CITY

GEOTECHNICAL INVESTIGATION REPORT

for the

New Recreation Facility and Aquatic Center Stand

Brentwood Family Aquatic Complex

195 Griffith Lane

Brentwood, California

September 16, 2004

Prepared By:

4LEAF, INC.

Prepared for:

City of Brentwood
Engineering Department

4LEAF, INC.

September 16, 2004
4LEAF Job Number: J148

Ms. Marna Huber
City of Brentwood Engineering Department
708 Third Street
Brentwood, CA 94513

Subject: Geotechnical Investigation Report for the New Recreation Facility and Aquatic Center Stand, Brentwood Family Aquatic Complex, Brentwood, California

Dear Ms. Huber:

In accordance with your authorization, 4LEAF, Inc. (4LEAF) has performed a geotechnical investigation at the subject site located in Brentwood, California. The enclosed report provides a description of the field investigations performed at the site on July 2 and July 12, 2004, and presents geotechnical recommendations for design and construction of foundations, slabs-on-grade, retaining walls, and earthwork for this project.

In summary, it is our opinion that:

- The existing soil conditions beneath both planned building areas have a very significant potential for damaging collapse and/or time-related settlement if they are heavily loaded. Thus, ground improvement is necessary for building support and management of settlement or subsidence beneath both building locations.
- The presence of at least 50 feet of stiff, fine-grained soils and very minor thicknesses of medium dense to very dense sands beneath the project area, subject to ground movement, are amenable to shallow-type foundations.
- The presence of moderately expansive soils in the near-surface soil profile is typical of the Brentwood area. Such soils will influence the performance of paving and minor shallow foundations and will require appropriate attention to detail in design and construction.
- The depth to groundwater, being approximately 30 feet, will not influence construction. Further, the depth to groundwater, combined with the very minor presence of clean and loose sands suggests a very low potential for seismic liquefaction.

- The presence of moderately expansive soils in the near-surface soil profile is typical of the Brentwood area. Such soils will influence the performance of paving and minor shallow foundations and will require appropriate attention to detail in design and construction.

- The presence of moderately corrosive soils in the near-surface soil profile is typical of the Brentwood area. Such soils will influence the performance of buried metallic items such as piping and will require appropriate attention to detail in design and construction. Soil chemicals present will not necessitate the use of special types of cement in concrete.

The conclusions and recommendations presented in this report are based on limited subsurface geotechnical exploration and laboratory testing programs. Consequently, variations between anticipated and actual subsurface soil conditions may be found in localized areas during construction. If significant variations in the subsurface conditions are encountered during construction, 4LEAF should be requested to review the recommendations presented herein and provide supplemental recommendations, if necessary.

In addition, design plans and specifications should be reviewed by 4LEAF prior to their issuance to ensure conformance with the general intent of our recommendations presented in the enclosed report.

4LEAF appreciates the opportunity to perform engineering services for the City of Brentwood on this important project. If you have any questions concerning the information presented in this report, please contact us at (925) 462-5959.

Sincerely,
4LEAF, Inc.

REGISTERED PROFESSIONAL ENGINEER
GENE A. BARRY
No. C 64174
Exp. Jun-07
CIVIL
STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER
JOHN R. SUTTON
LIC No. GE 812
EXP. 12/31/06
GEOTECHNICAL
STATE OF CALIFORNIA

Gene A. Barry, P.E.
Principal Engineer

John R. Sutton, G.E.
Geotechnical Engineer

TABLE OF CONTENTS

TABLE OF CONTENTS (continued)

FIGURES

Figure

APPENDICES

Appendix

1.0 INTRODUCTION

4LEAF, Inc. (4LEAF) was contracted by the City of Brentwood Engineering Department to provide geotechnical engineering services for the investigation phase of the proposed new recreation facility and aquatic center concession stand and the Brentwood Family Aquatic Complex in Brentwood, CA. This report presents the results of the geotechnical investigation for the new recreation facility and aquatic center concession stand in Brentwood, CA. A site vicinity map showing the site location is presented in Figure 1.

1.1 PROJECT DESCRIPTION

The new recreation facility will be located southeast of the existing Brentwood Family Aquatic Complex (aquatic center building and three swimming pools). The concession stand will be a separate building that is located immediately southeast of the aquatic center building. A new parking lot area will bound the two buildings to the south and east. Topography at the site is relatively flat. Figure 2 shows the locations of the existing and proposed buildings.

The City of Brentwood has retained the services of Quattrocchi Kwok Architects and ZFA Structural Engineers in Santa Rosa, CA for the building's design. Based on our phone conversations with the project's architect and structural engineer, the planned recreation facility building, with plan dimensions of approximately 150-foot length and 40-foot width, will be a single story, pre-engineered, metal-framed building with top plates at 12 and 22 feet above the floor level. The building will have 7½-foot-high masonry perimeter walls. The concession stand will be a masonry-walled building, approximately 25 feet by 20 feet in width, with its roof extending approximately seven feet on each side of its narrow dimension. The walls' top plate will be approximately 12 feet above the floor level. The roofs for both buildings will be conventionally pitched and framed. The floors in both buildings will be slabs-on-grade.

Because the new recreation facility and concession stand buildings will not house "Essential Services" such as police and fire dispatch, they are not being designed to the specific requirements of such a building as described in the California Building Code. The applicable building code is the 1997 Uniform Building Code (UBC), as adopted by the City of Brentwood's chief building official.

1.2 PURPOSE AND SCOPE OF SERVICES

The purpose of this geotechnical investigation is to investigate the surface and subsurface conditions at the site and provide recommendations for design and construction of foundation and related earthwork. The scope of services, as outlined in our work plan dated June 10, 2004, included:

- Reviewing available subsurface data in the geotechnical investigation performed in March 1999, for design and construction of the adjacent Brentwood Family Aquatic Complex.
- Advancing three (3) cone penetrometer test (CPT) probings and drilling and sampling two (2) hollow-stem auger (HSA) borings.
- Performing geotechnical and corrosion laboratory testing of selected samples.
- Performing engineering analysis of the data.
- Preparing a report that summarizes the findings of the geotechnical investigation, conclusions, and recommendations for foundation design and earthwork-related activities.
- Collecting characterization samples from the drill cuttings generated from the HSA borings to generate a disposal profile with a landfill facility.

2.0 SITE INVESTIGATION

The following sections summarize the reconnaissance, subsurface investigation, and laboratory testing that were performed in support of the project.

2.1 SUBSURFACE INVESTIGATION

John Sutton with 4LEAF performed a site reconnaissance visit on June 16, 2004, to locate and mark the proposed boring locations. The field investigation was performed over two phases so that 4LEAF's geotechnical engineer could review the CPT probing data and identify issues such as potential liquefiable zones prior to locating and drilling the HSA borings.

Phase I of the field investigation was performed at the site on July 2, 2004. It consisted of advancing CPT probes at three (3) locations in the vicinity of the recreation facility building to depths of approximately 50 feet below ground surface (bgs). The three CPT probings (designated 4L-CP1 through 4L-CP3) were drilled at the approximate locations shown on Figure 2. A CPT probe could not be advanced near the planned concession stand because the area was not accessible by the CPT rig due to soft and wet soil conditions in the grassed area surrounding the planned building location.

Following a review of the Phase I CPT data, Phase II of the field investigation was performed on July 12, 2004, and consisted of drilling HSA borings at two (2) locations (Figure 2). Boring 4L-B1 was drilled to a depth of 40 feet bgs at a location adjacent to CPT probing 4L-CP1. Boring 4L-B4 was drilled to a depth of 45 feet in the area of the concession stand building. The HSA drill rig was able to access the proposed concession stand site by laying sheets of thick plywood over the lawn surface to distribute the load and minimizing tire rutting.

Prior to the start of our field investigation, 4LEAF contacted Underground Service Alert (USA) to notify underground utilities at the proposed CPT probing and HSA boring locations. A drilling permit for the project was also obtained from the Contra Costa County Environmental Health Department.

4LEAF's geotechnical engineer was on site during both phases of the investigation to provide technical direction during the probing operation, direct and continuously log the soil borings, collect samples from the borings, and measure groundwater levels. CPT probings 4L-CP1 through 4L-CP3 were performed by Gregg Drilling of Martinez, California using a truck-mounted, 20-ton CPT rig. HSA borings 4L-B1 and 4L-B4 were also drilled by Gregg Drilling using a mobile model B61, truck-mounted drill rig using 8-inch outside diameter (OD), 3½-inch inside diameter (ID), continuous flight, hollow stemmed augers. The three CPT probings were advanced to a depth of 50 feet bgs to collect data to evaluate the potential liquefaction concerns at the site. Each CPT and HSA boring location was cleared for shallow obstructions by hand-augering to 6 feet bgs. For CPT probings, each hole was then partially backfilled prior to commencing the probing in the same location.

Upon their completion, the probings and borings were backfilled with cement grout in accordance with the County's requirements. A representative from the Contra Costa County Environmental Health Department was on site to observe grouting activities on both days. Soil cuttings and excess grout generated during the drilling and backfilling of the CPT probings and HSA borings were placed into four, 55-gallon drums. 4LEAF personnel collected a composite soil sample from the drums and submitted the sample to McCampbell Analytical Laboratory in Pacheco, CA. The sample results were used to profile the drill cuttings for waste disposal. The sample was analyzed for Title 22 metals, total extractable hydrocarbons as diesel (TEH-d), total volatile hydrocarbons as gasoline (TVH-g), volatile organic compounds (VOC), and semi-volatile organic compounds (SVOC).

2.1.1 Cone Penetrometer Testing

Each CPT probe was sounded using an integrated electronic cone system. The soundings were conducted using a 20-ton truck-mounted rig, hydraulically advancing the standard cone with a tip area of 15 square centimeters (cm^2) and a friction sleeve area of 225 cm^2. The standard cone has an equal end area friction sleeve and a tip area ratio of 0.85 (Gregg, 2004).

The cone recorded measurements of cone bearing capacity (qc), sleeve friction (fs), and dynamic pore water pressure (u2) at 5-centimeter (cm) intervals during penetration to provide a nearly continuous hydrogeologic log. The recorded cone measurements are stored on disk for further analysis and reference. Logs for CPT probings 4L-CP1 through 4L-CP3 are presented in Appendix A. The cone also contains a porous filter element located directly behind the cone tip that is used to measure dynamic pore pressure variation as the cone is advanced, but more usefully, for porewater pressure dissipation tests (PPDT) during appropriate pauses in penetration. Prior to penetration, the filter element is fully saturated with silicon oil under vacuum pressure to facilitate accurate and fast dissipation.

Pore Pressure Dissipation Tests

The PPDTs were conducted at various intervals in CPT borings 4L-CP1 and 4L-CP3 to measure hydrostatic water pressures and determined the approximate depth of the groundwater table. A PPDT was conducted when the cone was halted at specific intervals determined by 4LEAF's on-site geotechnical engineer. The variation of the penetration pore pressure (u) with time is measured behind the tip of the cone and recorded in the CPT rig computer system.

Pore pressure dissipation data can be interpreted to provide estimates of equilibrium piezometric pressure, phreatic surface, and in situ horizontal coefficient of permeability (kh). Results of pore pressure dissipation tests for CPT probings 4L-CP1 and 4L-CP3 are presented in Appendix B.

Seismic Cone Penetrometer Testing

Seismic cone penetrometer testing (SCPT) was performed on CPT boring 4L-CP2. A seismometer within the cone tip was utilized to measure compression and shear wave velocities at 5-foot depth intervals while advancing probe 4L-CP2. At each 5-foot depth increment, a steel beam was forced onto the ground surface by truck-mounted hydraulic rams and then an electric hammer was used to strike the end of the beam. The amplitude and wave frequency due to the strike was measured at each depth interval. The probe was then advanced the next five feet, during which the normal CPT parameters were collected. Following completion of the probing to the 50-foot total depth, the spectra from each depth increment were summed and integrated to result in an individual shear wave velocity for every five-foot depth interval. Results of the seismic testing for CPT probing 4L-CP2 are presented in Appendix C.

2.1.2 Hollow-Stem Auger Drilling and Testing

Borings 4L-B1 and 4L-B4 were continuously logged by 4LEAF's geotechnical engineer and soil samples collected from selected intervals. The soils from the boring were visually classified using the Unified Soil Classification System (USCS), ASTM D2488. The boring logs for 4L-B1 and 4L-B4 are presented in Appendix D.

Cuttings from hand auger drilling were recovered from between one and six feet depth in the two HSA borings collected as a bulk sample for and tested for soil moisture/density relationship. Similarly, soil cuttings from the upper two feet of the hand-augered pre-boring at 4L-CP1 was collected as a bulk sample and tested for the soil's corrosion potential.

Soil samples were collected at sampling intervals of 5-foot depth increments (typical) and were recovered by driving a tubular soil sampler into the soil using a 140 pound hammer raised 30 inches by a hydraulic, down-hole trip-hammer, until 18 inches of sampler penetration was achieved, generally following the method of the Standard Penetration Test (SPT), ASTM method

D1586. Samples of the subsurface soils were obtained using either the standard SPT sampler (2-in. OD and 1⅜ in. ID), or the Modified California sampler (3.0-in. OD and 2.5-in. ID), as selected in the field by 4LEAF's engineer. Shelby tubes were not used for sampling because the soils were determined to be too stiff for Shelby tube sampling as estimated from the CPT or auger drilling resistance.

The number of hammer blows required for each 6 inches of penetration was logged, and the total blow count for the last foot of penetration logged as the "N-value". After conducting the SPT test, the auger is then advanced to the next sampling depth increment with the sampler still in the ground.

The California sampler is lined with three or more 2½ -in. nominal diameter, 6-in. long, thin-walled, metal sleeves which provides soil specimens in relatively intact and undisturbed condition, suitable for laboratory testing and measurements. The SPT sampler is used without liner sleeves because it provides a more accurate indication of soil firmness for granular soils for use in analyses such as for liquefaction but does not yield the intact, and relatively undisturbed samples needed for some of the laboratory tests.

As the sampler is retrieved, the field engineer observes the drill rod string and sampler for the indication of groundwater. After the sample barrel is recovered from the boring, the barrel is opened and examined, first for presence of groundwater, if any, and secondly for recovery in regard to the 18 in. driven. With the SPT sampler, the soil is classified and some of the soil collected in a labeled, zip-lock type bag. With the California sampler, the liner sleeves are carefully separated using a knife, and the sample ends inspected for recovery and soil classification. Tight-fitting plastic caps are set on each end of the lowest sleeve (and sometimes also the second), the tube(s) uniquely labeled, and stored in a cool place to await shipment to the laboratory. The soil in the sampler tip and the other sleeves is then extracted by the field engineer, examined, visually classified (ASTM D2488), and documented on the drilling log. More liner sleeves are placed in the barrel, and the sampler lowered down through the hollow auger stem to the bottom of the boring, ready for the next penetration test. At that time, the boring depth is checked for heave by measurement of sampler rod length against the auger length.

Following completion of each boring, and prior to auger removal, depth to groundwater was checked using a heavily weighted cloth tape. The borings were then grouted to the surface by tremie injection of fluid cement-bentonite within the auger stem as the auger was being retracted.

2.1.3 Groundwater Measurements

Depth to groundwater was directly measured in HSA borings 4L-B1 and -B4, using a heavily weighted cloth tape measure. Depth to groundwater was also interpreted from the CPT probings using the porewater pressure cell in the CPT tip.

2.2 LABORATORY TESTING

Laboratory tests were performed on selected soil samples collected from HSA borings 4L-B1 and 4L-B4 to evaluate appropriate physical and chemical characteristics and geotechnical engineering properties. Soil classifications made in the field were changed as appropriate, based on the laboratory test results. The description of the lithological conditions and classifications are presented on the logs for borings 4L-B1 and 4L-B4.

Laboratory testing was performed by Cooper Testing Laboratory of Mountain View, California and included the measurement of field moisture content (ASTM D2216) and unit weight (ASTM D2937), strength in direct shear (ASTM D3080), constant rate of strain consolidation (ASTM D4186), Atterberg (plasticity) limits (ASTM D4318), measurement of collapse potential (ASTM D5333), specific gravity (ASTM D854m), and #200 sieve wash analysis (ASTM D1140). A bulk sample collected from hand auger cuttings recovered from probing 4L-CP1 was tested for the measurement of soil corrosivity potential (Caltrans standard package). In addition, a composite of cuttings from the upper six feet of the two HAS borings was tested for moisture-density relationship (compaction curve) by ASTM Method D1557-00 at Construction Testing Services (CTS) of Pleasanton, California. The laboratory results are presented on the boring logs for 4L-B1 and 4L-B4. Copies of laboratory test reports from Cooper Testing Laboratory and CTS are presented in Appendix E.

3.0 SITE CONDITIONS

The following sections discuss the topography, climate, and drainage at the site, the regional and local geology, local faulting and seismicity, and subsurface conditions at the site.

3.1 TOPOGRAPHY, CLIMATE, AND DRAINAGE

The site area is essentially level, at approximately 83 feet elevation relative to mean sea level (msl). The recreation center site is generally south of, and adjacent to, and encroaches upon the Brentwood Family Aquatic Complex, that comprises the aquatic center building and three swimming pools, surrounding landscaping and lawn areas, sidewalks, and paved parking areas. The City is currently completing construction of a large regional park on the quarter section of land immediately to the north and west of the existing aquatic center.

Average annual rainfall in Brentwood is approximately 12 inches, with about 80 percent of the rain falling between the months of November and April. This amount of rainfall is low compared to the San Francisco Bay area and Contra Costa County, which receives 40 inches near Orinda and 22 inches in Richmond. The area, typical of much of Contra Costa County, has net-negative evaporation; i.e. the annual rate of evaporation exceeds annual precipitation. Therefore, substantial additional artificial moisture must be applied (irrigation) to support lawns and coastal vegetation.

Site drainage will be directed to existing drainage inlets constructed by the City. These drain to underground piping beneath Balfour Road and Griffith Lane. Currently, rainfall that falls directly on the open area where the planned recreation facility will be constructed, accumulates on site, infiltrates into the bare ground, and/or becomes sheet flow to the drainage inlets.

Completion of the planned recreation facility building and concession stand will increase runoff in the southern portion of the site because of increased roof, sidewalk, and pavement cover. The planned buildings will need to be efficiently drained so that newly concentrated water cannot cause sedimentation or erosion during or after development.

3.2 REGIONAL GEOLOGY

Brentwood lies on near-level land to the northeast of hills that flank Mount Diablo. These level lands are Holocene–aged sediments, less than 10,000 years old, and include soils, the alluvium of present streams, fans deposited at the mouths of canyons in the hills, and deltaic sediments of the Sacramento/San Joaquin River Delta. The Holocene materials are only a few tens of feet thick at most. Underlying these recent deposits is a sequence of sediments that entirely fill the Meganos Canyon. The lithology depends on nearby source materials (Crane 1995). The hills that bound the Brentwood bottom land to the west and south include Quaternary-aged terrace deposits, Pliocene and Pleistocene-aged gravels, and sediments of the Great Valley sequence, and include coal and silica deposits. Hills in the west part of the city and to the north are sand dunes.

Oil and gas was recovered from production wells 3,500 to 4,000 feet deep in the Brentwood Oil and Gas Field, located on about 1,000 acres at the western city limits. At this time, production has essentially ceased. Gas was recovered from wells at about 8,000 feet depth in the East Brentwood Gas Field, however, production ceased in the early 1990s (Crane, 1995). Individual agricultural water wells, completed at various depths, supply the agricultural belt. Many of the shallower wells have been technically abandoned as Brentwood became urbanized. Some of the water wells are known to have high nitrate content. No land subsidence is known nor expected in the site area because of water or mineral extraction.

3.3 LOCAL GEOLOGY

The Quaternary Geologic map of Contra Costa County (Helley and Graymer, 1997) indicates that soil underlying the site is Holocene-aged alluvial fan and fluvial deposits. These soils were likely deposited as alluvial and fluvial deposits that originated in the hills and mountains to the west and south of Brentwood (Figure 3). These alluvial and fluvial deposits are described as loose, moderately- to well-sorted sandy or clayey silt grading to sandy or silty clay. Flash flooding events transported these deposits from their sources. As a result, the sediments are generally porous and permeable. They are susceptible to collapse when loaded and saturated (Gibbs and Bara, 1962). Sedimentary deposits beneath the site increase in firmness with depth. Due to their relative depth, bedrock-like materials will not influence design or performance of the project.

3.4 LOCAL FAULTING AND SEISMICITY

The project site, like the entire Bay Area, is located within a seismically active region. The nearest documented active faults are the Concord fault, mapped approximately 25 kilometers (km) (15 miles) to the northwest, and the Greenville-Marsh Creek-Clayton fault system, and the Calaveras fault, which are mapped approximately 14 km (9 miles) to the southwest, respectively (Terrasearch, 2001). The Type A source Hayward and San Andreas faults are mapped 42 km (26 miles) and 74 km (46 miles) southwest of the site, respectively.

Several deep buried thrust faults, such as the Mount Diablo and San Joaquin fault are other faults present in the vicinity of the site. The Coast Ranges/Great Valley blind thrust fault, responsible for the 1983 Coalinga earthquake, has been postulated along the western edge of the Central Valley, and within a mile of the site (Carey, 1992). However, the California Geological Survey (CGS) currently has deleted "Great Valley Section 6" (which would be the section of the fault passing through Brentwood) from its published Alquist Priolo Earthquake Zoning Act in favor of the Mount Diablo Thrust fault (CGS, 2004a). The Brentwood, Kellogg, Vaqueros, Davis, and Camino Diablo faults, collectively grouped as the "Vaqueros faults" were studied in great detail during the planning of Los Vaqueros Reservoir, and were determined to be inactive (Biggar and Wong, 1992). Likewise, the Antioch fault was demonstrated to be inactive (Wills, 1992). Crane's map of the Brentwood quadrangle shows the Sherman Island and Midway faults. These faults are not considered to be active by the CGS under the Alquist-Priolo Earthquake Fault Zoning Act.

The site is not located within an area where seismic-related ground breaks are expected (see reference to CGS in the preceding paragraph). It should be recognized that the site will probably experience moderate to severe ground shaking when an earthquake occurs on one of the many active faults in the Bay Area.

3.5 SUBSURFACE CONDITIONS

The great majority of soils encountered in the borings and probings were, interlayered, clayey silt and/or silty clays, light brown in color. As shown on the boring logs, these fine-grained materials were often sandy.

3.5.1 Shallower Soils

In the recreation center site, soils in the upper seven-or-so feet were dessicated and exhibited low unit weight as compared to the moisture–density test relationship curve and to the charts prepared by Gibbs (Gibbs and Bara, 1962). Deeper soils indicated increasing moisture content and unit weight with depth. While blowcount (N-value) and CPT resistance indicated the clays and silts were stiff to very stiff, the low unit weights of shallower soil samples are indicative of collapse-prone soils.

Approximately half of the area where the recreation building will be constructed has been topped with excess soil that originated in the adjacent park. These soils were relatively dry and of low firmness.

Laboratory tests indicated that the site clays in boring 4L-B1 have a liquid limit (LL) of 31 and a plasticity index (PI) of 11. Site clays in boring 4L-B4 have a LL of 40 and a PI of 22. A direct shear test (consolidated undrained) on a sample of sandy silt collected in boring 4L-B4 from approximately five feet bgs resulted in a friction (Ø) angle of 17.2° and cohesion intercept of 240 pounds per square foot (psf). A consolidation test conducted on a sample from boring 4L-B4 at approximately 10 feet depth under constant rate-of-strain conditions with natural saturation of 90 percent indicated a preconsolidation pressure of 2,000 psf. A collapse test conducted on a sample of sandy silt from boring 4L-B1 at approximately five feet depth, having an initial dry unit weight of 83 pound per cubic feet (pcf), collapsed 14 percent after inundation under a load of 4,400 psf.

3.5.2 Deeper Soils

Soils below about 8 feet depth indicated moderate unit weight. Moisture contents in boring 4L-B1 ranged from 11 to 15 percent in the upper 12-or-so feet, while soils from the same depth range in boring 4L-B4, located in the irrigated lawn area, indicated near-saturation moisture conditions in the range of 19 to 25 percent. Below about 15 feet depth, and to the groundwater depth, moisture contents in the two borings were similar, in the range of 16 to 20 percent.

Sand zones were indicated by two of the CPT probings in the recreation center building area. In 4L-CP2, these sand zones totaled 6 feet thickness in the 30- to 40-foot depth range, and in 4L-CP-3 a sand layer 3 feet thick was indicated between 40 and 43 feet depth. All these sands had tip resistances indicative of dense to very dense soils.

3.5.3 Groundwater

Depth to groundwater was directly measured in borings 4L-B1 and 4L-B4 at 32 and 30.5 feet bgs, respectively shortly after drilling. These approximated the depths of saturated soil indicated in samples recovered from the two borings. Depth to groundwater was also determined from pore water pressure dissipation tests in CPT probings 4L-CP1 and 4L-CP3, which resulted in groundwater depths of 30.7 and 31.4 feet respectively. On this basis, groundwater depth is approximately 31 feet bgs.

The irrigation-caused shallow ground saturation identified in the boring located at the planned concession stand building may have resulted in the formation of perched groundwater lenses at shallower depths and should be considered in planning for excavation and trenching in the lawn-covered areas. The same saturated condition situation should be expected adjacent to the irrigated lawn areas on the north side of the planned recreation center building site. The drilling and penetration would have influenced the measured and interpreted depths to groundwater, and if allowed to stabilize over time, may be different from those reported. Further, the groundwater level can fluctuate depending on factors such as seasonal rainfall, groundwater withdrawal, and construction activities on this and adjacent properties. Groundwater levels may rise several feet during a normal rainy season. However, to groundwater levels are unlikely to rise sufficiently to change its effect on site geotechnical or construction conditions.

3.5.4 Shear Wave Velocity

Shear wave velocity measurements were made at 5-foot depth intervals between 10 and 50 feet bgs in CPT probing 4L-CP2. From those measurements, the shear wave velocity of soils beneath the site was interpolated for each 5-foot depth interval of the profile. The lowest shear wave velocity, 600 feet per second (ft/sec) was measured for the interval between 10 and 15 feet bgs, the shallowest depth range measured. Shear wave velocity measurements below 20 feet depth approximated 900 ft/sec. The shear wave velocity profile with depth is in provided in Appendix C.

3.5.5 Corrosion

The suite of corrosion tests performed on a composite sample of the fill placed on the site, which is understood to have been derived from excess soil from grading for the adjacent City park, yielded results typical of soil in the Brentwood area. As generalized in the County soil report (Welch, 1977), the test results indicate that the local soils have moderate potential be corrosive to buried metal objects, such as piping. The test results do not indicate the presence of ground constituents which would, according to Section 1904 and Tables 19-A-1 through 19-A-7 of the 1997 UBC, necessitate the use of special cement for buried concrete. The suite of tests conducted was in accordance with regional practices, and are those required by Caltrans in analysis of similar projects. 4LEAF does not have expertise in corrosion engineering. If further information is needed in regard to soil corrosivity, a professional corrosion engineer should be consulted.

3.5.6 Commentary

The above is a general description of soil and groundwater conditions encountered in the three CPT probings and two HSA borings drilled for this investigation. More detailed descriptions of the soil and groundwater conditions encountered are provided in the CPT and HSA boring logs included in Appendices A and D, respectively.

Soil and groundwater conditions can deviate from those conditions encountered at the probing and boring locations. If significant variations in the subsurface conditions are encountered during construction, 4LEAF should be notified immediately, and it may be necessary for us to review the recommendations presented in this report and recommend adjustments as necessary.

4.0 DISCUSSION AND CONCLUSIONS

The primary geotechnical considerations for the planned buildings at the site are:

1. The shallow soil soils that underlie the planned building areas have a very significant potential to collapse and/or undergo time-related consolidation if they become saturated after being loaded by the weight of the buildings. Based on our observations of the adjacent development and our understanding of the final project configuration, this a highly likely scenario. In order to minimize the effect of this potential subsidence or settlement beneath the two building locations, it will be necessary to strengthen the soil, a process known as "ground improvement". While soil recompaction will densify the soil matrix to increase its firmness and strength, the ground improvement process also includes several related ground strengthening processes that would be performed concurrently with the soil densification to minimize the collapse potential and enhance foundation stability for the building sites. Ground improvement is discussed below in Section 5, Recommendations.

2. The presence of at least 50 feet of stiff, fine-grained soils and very minor thicknesses of medium dense to very dense sands beneath the project area, subsequent to ground improvement, are amenable to shallow-type foundations.

3. The depth to groundwater, being approximately 30 feet, will not influence construction. Further, the depth to groundwater, combined with the very minor presence of clean and loose sands suggests a very low potential for seismic liquefaction.

4. We conclude that there is little likelihood of seismic soil liquefaction at this site. This is due to both the significant depth to groundwater beneath the ground surface and the medium dense to dense consistency of the majority of the sands present near or beneath the groundwater depth.

5. The presence of moderately expansive soils in the near-surface soil profile is typical of the Brentwood area. Such soils will influence the performance of paving and minor shallow foundations and will require appropriate attention to detail in design and construction.

6. The presence of moderately corrosive soils in the near-surface soil profile is typical of the Brentwood area. Such soils will influence the performance of buried metallic items such as piping, signage supports and ground anchors, and will require appropriate attention to detail in design and construction. Soil chemicals present will not necessitate the use of special types of cement in concrete in accordance with the guidance in Section 1904 and Tables 19A-1-1 through 19A-1-7 of the 1997 UBC.

Additional discussions of the conclusions drawn from our investigation, including general recommendations, are presented below. Specific recommendations regarding geotechnical design and construction aspects for the project are presented in Section 5, Recommendations.

4.1 FOUNDATIONS

4.1.1 Foundation Types

As described above, the existing soil conditions beneath both planned building areas have a very significant potential for damaging collapse and/or time-related settlement if they become saturated after they are subjected to building loads. As a result, ground improvement is necessary for building support and management of settlement or subsidence beneath both building locations.

We considered the appropriateness of several available ground improvement technologies in regard to: (a) the present site conditions and (b) the adjacent, recently completed aquatic center with its high level of public usage. We concluded that the best solution would be a combination of shallow-depth ground improvement with conventional, spread- and strip-type foundations.

The most appropriate and economical types of foundations for the proposed structure are spread-type footings and mats. Spread-type footings include isolated footings for single columns and linear type (strip) footings for multiple columns or walls. Mat or raft footings occupy the entire building footprint. Mats tend to be more economical when high column loads result in individual foundations that occupy more than about half the site area. Mat foundations become more economical because footing formwork is minimized, and because reinforcing steel is continuous and eliminating most of the need for bent or hooked bars. Mat foundations help distribute the foundation load over a larger area, therefore, the magnitude of settlement, both total and differential, is reduced. By increasing mat stiffness, differential settlements can essentially be eliminated. However, column loadings for the proposed buildings will be relatively low and distantly spaced. Thus, it is not expected that mat foundations will be as costeffective as conventional spread footing foundation systems. For this reason, we have not provided criteria for designing mat foundations but would be happy to do so if requested.

Conventional spread footings should bottom on engineered fill that has been improved by the ground improvement process discussed below in Section 5, Recommendations. While spread footings will be more susceptible to variations in soil firmness and soil type than mat footings, it is our opinion that, following ground improvement, conventional spread footings can be designed to provide normal settlement and acceptable building performance.

4.1.2 Design of Conventional Spread Footings

We recommend founding the buildings on conventional spread and/or strip foundations that bottom at least 18 inches below finished site grade. Foundations should be supported on no less than three feet thickness of engineered fill in which there is a "geogrid sandwich" as described below in Section 5.1, Site Preparation.

Continuous (strip) foundations should be no less than 18 inches wide and should be reinforced with no less than two, #4 bars continuous in the base and one in the stem portion. Spread footings should be no less than 24 in. square and should be reinforced with no less than three, #4 bars in each direction. Such foundations may be designed using an allowable design load (dead load plus normal live load) of 2,000 psf. This value may be increased by one third for short-term live loads, such as wind or seismic loads.

Passive resistance, where there is at least 10 feet of level or up-sloping soil adjacent to the footing, may be taken at 210 pcf-equivalent fluid pressure (efp) for foundation sliding or seismic reaction, and/or use sliding friction of 0.25 times dead load when footings are on level, engineered fill.

4.1.3 Foundation Settlements

Spread footings designed using the above recommended site preparation and design criteria may be expected to settle less than one inch. Differential settlement across the width of the recreation center building may be expected to be half of this amount. Settlement of the concession stand building on a similarly prepared pad are likely to be of the order of one inch, of which less than half may be differential.

4.2 SLABS ON GRADE

Slabs on grade should be supported on at least four inches of granular fill (drain rock or aggregate base) as a capillary break. At this site, we recommend taking additional care to ensure the integrity of the moisture barrier. It should be no less than 15 mil thickness and protected by a sand layer no more than two inches thick.

Slabs on grade and minor paving around the building should be designed for the heave-induced stresses due to shrink-swell behavior of local soils as they desiccate and later absorb moisture. Slabs should be supported on no less than one foot of subgrade soil recompacted as engineered fill as discussed below in Section 5, Recommendations.

4.3 DEMOLITION AND GRADING

It is likely that the majority of soil on the two building sites will be suitable for re-use as engineered fills, however, precedent use in agriculture and as a construction laydown area dictates the usual precautionary measures to discover old buried infrastructure. Near surface soils may include topsoil and deleterious materials that are not generally suitable for foundation or paving support. Any such materials discovered during excavation can be removed. Further, the 1½ foot thick prism of fill that has been stockpiled on the northwest half of the recreation center building site was not placed as an engineered fill and thus should not be used for building support in its current state. The soil itself appears to be suitable for re-use as engineered fill but its quality should be re-evaluated during the site preparation phase.

Soils beneath the concession stand site include landscape fill that should be wasted or stockpiled for such reuse. Underlying soils will be at elevated moisture content and likely will need to be dried appropriately before they can be re-used in engineered fills.

4.4 EXCAVATION

Excavation of the site soils should be relatively straightforward. Excavation side slopes should not be steeper than the California Occupational Safety and Health Administration (Cal/OSHA) maximum inclination of 45 degrees (1:1) for a Class B soil. Soils beneath presently irrigated areas may not, due to high moisture content, be stable if excavated to the OSHA maximum inclination. Consideration of the locations of existing aquatic center building and wall foundations, and resulting surcharge effects on excavation slope stability must be considered. Similarly, maintenance of adequate foundation support for buildings and masonry walls must be considered in designing excavations for site preparation. Additionally, recently placed underground utilities which parallel the southeast perimeter wall of the aquatic center appear to lie within the areas that will be needed for site preparation and would need to be relocated.

Because the shallowest groundwater measurement was approximately 30 feet bgs, groundwater would in general, not be expected to be a factor in excavation safety and stability. The irrigation-caused shallow ground saturation identified in the vicinity of the planned concession stand building should be considered in any planning for excavations and trenching in and adjacent to all areas that have been irrigated.

4.5 GEOLOGIC AND SEISMIC HAZARDS

As stated in Section 3, the site is not located near any active faults according to the State Geologist. Therefore, no ground break hazards exist for the site. There are also no slopes in the site vicinity that would endanger the completed facility due to seismic shaking. Nevertheless, the region is seismically active. The alluvial fan and fluvial deposits of Holocene geologic age that underlie the site will amplify seismic shocks. A moderate to significant earthquake anywhere in the Bay Area or local region is likely to cause significant ground shaking at the site.

Seismic design criteria and factors based on UBC requirements are included in Section 5, Recommendations.

4.5.1 Seismic Shaking

The geologically recent soils of the Brentwood area will amplify seismic shocks. The California Geological Survey's Probabilistic Seismic Hazards Mapping program's Ground Motion Page provides the following acceleration data (10% probability of being exceeded in 50 years, or 475 year return interval) for the site:

	Ground Motion, %g for Alluvium-founded Sites
Peak Ground Acceleration, (Pga)	0.397
Spectral Acceleration (Sa) = 0.2 sec	0.966
Spectral Acceleration (Sa) = 1.0 sec	0.457

We conclude that the potential for seismic-caused, damaging soil liquefaction that could affect these buildings is low. Note, however, that the site soils will amplify the ground acceleration above the Pga. We find no reason, however, to depart from the seismic design requirements of the UBC / California Building Code (CBC).

Based on the shear wave velocity measurements in our field investigation, the soils at the recreation center site should be classified as S_D, in accordance with the 2001 CBC / 1997 UBC. The following seismic design factors apply to the site structures in consideration of the above source types and distances:

Seismic Zone	**§1629.4.1/Fig 16-2**	**Z = 0.40**	**Zone 4**
Source Type	Table 16U	Type A, B	Calif. Geol. Survey
Near-Source Factor Na	Table 16S	Na = 1.0	Source distances: A: 42 km; B: 9 km
Near-Source Factor Nv	Table 16T	Nv = 1.0	Soil type S_D
Seismic Coefficient Ca	Table 16Q	Ca = 0.44	Ca = 0. 44 x Na
Seismic Coefficient Cv	Table 16R	Cv = 0.64	Cv = 0. 64 x Nv

4.5.2 Liquefaction

Soil liquefaction occurs when loose, saturated, granular, and certain fine-grained deposits change from a solid to a liquid state during seismic shaking. Cyclic loading, i.e. stress reversal during an earthquake, increases inter-granular pore pressures and sufficiently violent shaking can cause some soils to soften. Significant damage resulting from liquefaction was experienced during the major earthquakes of Alaska and Nigita, Japan in 1964 and the Kobe, Japan earthquake in 1994. International committees of geotechnical engineers have studied liquefaction and prepared consensus reports that provide state-of-the practice analysis guidelines. The most recent report issued was the NCEER committee headed by Youd and Idriss, 2001.

Soil liquefaction is caused by multiple cycles of stress reversal. Long-duration earthquakes transmit multiple cycles of low amplitude wave motion for great distances from the source, causing liquefaction. Intense, short-duration earthquakes may not generate enough reversal cycles to cause liquefaction near the source. The 1989 Loma Prieta earthquake caused liquefaction of San Francisco's Marina District, located more than 100 km (62 miles) from the source. The Marina District is only a few feet above sea level and liquefaction generated sand boils, accompanied by three- to four-inches of settlement.

The NCEER procedure makes no recommendation on liquefaction of relatively confined layers at depth. It can be strongly argued that the sands identified deeper than 30 feet bgs in CPT probings are sufficiently confined by the overlying stiff clays and clayey silts so that the thickness of relatively impervious overburden soils will confine the active zone to prevent the mobilized water rising to the ground surface in order to dissipate (Pyke, 2003). This is a condition of liquefaction. The proposed ground improvement will additionally cause a "mat effect" that will confine soils beneath the buildings and should resist the effects of local, deep soil liquefaction, should it occur.

4.5.3 Landsliding and Lateral Spreading

As the site vicinity is level, seismic-caused land sliding will not affect the completed structures. As the site is laterally extensive, i.e. there are no irrigation canals or other "free faces" near the planned structure, the expectation of lateral spreading is also very low.

4.5.3 Densification

Densification due to a major seismic event is possible; however, it will affect the entire locality equally. The ground improvement we have proposed beneath the two planned buildings should minimize the effects of a seismic event.

4.5.4 Expansive Soils

The site soils, being clay-rich, will have moderate expansion potential based on their liquid limits. In addition, the Contra Costa County soil survey indicates surficial soils in the site area have high expansion potential. This expansion potential will need to be incorporated into design of near-surface facilities. However, conditions at the site are in general no less significant than in other areas of the county and can be managed with state-of the-practice design and construction details.

5.0 RECOMMENDATIONS

The following are recommendations for site preparation, foundation design, slab-on-grade and exterior flatwork, retaining walls, earthwork, pavements, drainage, and corrosion prevention. The existing soil conditions beneath both planned building areas have a very significant potential for damaging collapse and/or time-related settlement if they are heavily loaded. Thus, ground improvement is necessary for building support and management of settlement or subsidence beneath both building locations.

We considered the appropriateness of several available ground improvement technologies in regard to (a) the present site conditions and (b) the adjacent, recently completed aquatic center and its high level of public usage. We concluded that the best solution would be a combination of shallow-depth ground improvement with conventional, shallow-type foundations. Site preparation and foundation design and construction recommendations are presented in the following sections.

5.1 SITE PREPARATION

It is our understanding that the new buildings will be at approximately the same floor level as the adjacent pool building. As such, the grade for the proposed recreation facility building will need to be raised approximately 1½ feet. The 1 ½ feet of soil that has been recently placed over part of the planned recreation facility building should be reworked as described below.

5.1.1 General

In order to provide a stiff mat resistant that will distribute building loads to the soil uniformly and resist potential wetting-caused collapse, we recommend that each of the building sites be founded on a prism of structural geogrid-reinforced soil replacing the in-situ soils. To achieve this, each building site should be sub-excavated in an area that extends no less than eight feet outside each building perimeter, on all sides, and to a depth of at least three feet deeper than the foundation bottom elevation. The excavation bottom should be densified, and then backfilled with engineered fill, and sandwiched with several layers of structural geogrid as described below.

5.1.2 Clearing and Grubbing

Prior to the commencement of site grading, all deleterious material generated from the clearing of the site (e.g. all vegetative matter, paving, any foundation septic tanks, demolition debris, concrete, etc.) should be removed and legally deposed of off site. Tree removal should include removal of roots larger than a half-inch diameter and root masses. No plant material should be left on site where it might be placed in the fill during grading.

The fill currently on the northwest half of the planned recreation center building site should be removed and stockpiled for reuse. During the excavation process, any deleterious materials or other constituents found included therein should be culled and appropriately wasted.

Existing on-site utility lines should be relocated outside the work areas. All grading should be performed in accordance with Contra Costa County's general earthwork and grading specifications unless specifically revised or amended below, and in accordance with all applicable City of Brentwood Building Code requirements. All earthwork and grading operations should be performed under the direct observation and testing of the Soils Engineer.

5.1.3 Site Preparation

Areas to receive fill or paving as part of this project should be ripped to 12 inches minimum depth, and cross-ripped again at right angles to the first direction. The ripped bottom of the planned recreation center building excavation should then be thoroughly soaked over a period of two days.

The soil beneath the planned concession building is expected to be sufficiently moist, or may be too moist as will need to be dried prior to compaction.

The excavation bottom should then be compacted by no less than six complete passes of a heavy, vibratory, sheepsfoot-type compactor with a drum no less than six feet wide. Restore any depressions and over-excavate areas with native soils prepared and compacted as engineered fill, as described below.

5.1.4 Sub-Excavation

All sub-excavation should be performed under the general observation of the Soils Engineer. Sub-excavation should not extend close enough to adversely affect the integrity of adjacent structures. Should this condition be apparent, the Soils Engineer should be immediately notified so that we may clarify and/or modify our recommendations, as appropriate.

Excavated soils stockpiled for future use as backfill should be maintained to near optimum moisture. Stockpiles should be tarped to retain moisture and to prevent over-wetting. Overly wet soils may be dried by blending with drier soils. Overly wet soils should not be used in backfilling without prior drying.

5.1.5 Structural Geogrid

After densification, the excavation bottom, three feet deeper than the foundation bottom elevation, should be smoothed and then covered by a structural geogrid, equivalent in tensile strength and modulus to Tensar® BX1100. The excavation should then be backfilled in lifts to site grade with native or similar materials placed as engineered fill. Beneath each strip and spread foundation, sandwich two additional geogrid strips in the backfill, spaced at approximate one foot depth intervals beneath the foundation bottom. Extend these strips at least three feet beyond the foundation edges. Lap the sheets in accordance with manufacturer's recommendations but provide no less than two feet overlap or mechanical connections.

5.1.6 Engineered Fill and Backfill

Fill material should be similar to the native on-site soils, having an LL less than 42. Excavated on-site soil with an LL greater than 42 should be buried more than three feet below subgrade level (as engineered fill) or wasted off site. Engineered fill materials should be less than six inches in maximum size. Based on available soils data, we believe that the majority of on site native soil will be suitable for reuse. Engineered fill and backfill should be placed and compacted under the Soil Engineer's observation as described below. We recommend against backfilling foundation overexcavation with cementitious fill.

Prior to placing fill, the soils in areas to be filled should be thoroughly scarified, moistened, and then compacted. To ensure uniformity, all fill soil should be moistened and blended by thorough discing and/or mixing. On-site or similar materials should be moisture conditioned to at least two percent above the optimum moisture content per ASTM D1557. We recommend moisture conditioning of soil at the stockpile rather than at the point of placement.

Oversize materials (greater than three inches size) should be placed more than three feet away from foundation and trench areas, and not within a foot from paving subgrade level. Large rocks should transported off site or mixed into a soil matrix to minimize long term settlement. Do not place material larger than 1½ inches in size closer than six inches from geogrid layers, pipes or conduits.

Place the fill in eight-inch maximum loose lifts for "ride-on" compactors or four inches maximum for hand-operated compactors. Lifts shall be thick enough to ensure the geogrid layers are not harmed by the compactive effort. Make no less than four complete compactor passes per lift. For clays, use a sheepsfoot type roller with tamping feet no less than three inches long.

Ensure that water is available on site to wet the in-place soils and trench walls between lifts, especially in hotter weather. Uniformity of moisture content and firmness are most important. Clayey materials should be moisture conditioned to between two and five percent greater than the optimum moisture content according to ASTM D1557. Non-plastic materials including aggregate base (AB) may be placed at no less than the optimum moisture content.

Place the lifts horizontally, not on a slope; and compact from one end of the fill area to the other, completing each individual lift in a single effort before beginning the next. In warmer weather, re-moisten previously compacted fill prior to placing a new lift on top. Overly wet soils should be removed from the work area, dried back to the recommended moisture range prior to being re-used on site. When backfilling excavations below the design foundation elevation, bench the backfill into the pit walls at no steeper than a 1:1 slope ratio.

Engineered fill and backfill should be compacted to no less than 90 percent of maximum dry density according to ASTM D1557. AB should be compacted to 95 percent of maximum dry density (ASTM D1557). Fill and backfill placement and compaction must be performed under the observation of the Soil Engineer's on-site representative so that the documentation of Special Inspection required by the City's grading regulations could be provided.

5.1.7 Utility and Exterior Trenches

The utility designers must be made cognizant of the importance of the geogrid. Trenches must not penetrate the geogrid. If the geogrid is punctured, the area must be re-excavated and the geogrid's structural integrity restored under the guidance of the Soil Engineer.

All utility trench backfill must be compacted to a minimum relative compaction of 90 percent. No material greater than 1½ inches in size shall be placed in trench backfill within one foot of a pipe, conduit, or cable. Care should be exercised during fill placement to protect surface coatings and /or insulation from damage. Exterior trenches paralleling footings and/or extending below a 1:1 plane projected from the outside bottom edge of the footing should be backfilled with a minimum three-sack cement-sand slurry. Density testing, together with probing, should be conducted to verify the desired compaction of all trenches.

5.2 FOUNDATIONS

We recommend founding the buildings on conventional spread and/or strip footings that bottom at least 18 inches below finished site grade. Foundations should be and supported on no less than three feet thickness of engineered fill. Spread footings will settle an amount based both on their load magnitude and the foundation size. As variability of firmness of native alluvial site soils must be expected, some differential settlement between footings must be expected. Our foundation recommendations are presented below.

We considered the appropriateness of several available ground improvement technologies in regard to (a) the present site conditions and (b) the adjacent, recently completed aquatic center and its high level of public usage. We concluded that the best solution would be a combination of shallow-depth ground improvement with conventional, shallow-type foundations.

5.2.1 Square and Continuous Footings

Building and wall footings should be cast on consistently dense soil at least 18 inches below adjacent final grade. Square pad footings should be at least 24 inches square. Continuous footings should be a minimum of 18 inches wide or greater where required by Code.

Foundation bearing capacity will be controlled by settlement considerations. Square or continuous footings bearing on competent native soil or engineered fill, on at least three feet of engineered fill, may be designed using an allowable bearing capacity due to dead load plus normal live load of 2,000 psf. These design pressures may be increased by one third for load combinations that include wind or earthquake.

Foundations on native soil and on engineered fill that are designed according to the above criteria are estimated to result in settlement of less than one inch. All settlement may be assumed to be differential across footings and between footings.

All footing excavations should be inspected by the Soils Engineer to verify soil firmness prior to the placement of forms and reinforcing steel. If excavation bottoms need to be densified to achieve appropriate firmness, we recommend moisture conditioning then re-working the soils as recommended for engineered fill. Do not place pervious materials such as aggregate base or drain rock in excavation bottoms as they will collect water over time, which will encourage heave. To ensure that lateral resistance is achieved, footing concrete should be cast directly against intact soil or engineered fill.

As the native site soils are clay-rich, soils exposed in open foundation excavations should be maintained at no less than their optimum moisture content and with stable bottom until concrete is placed. This may entail covering the excavations with plastic sheeting in summer and tenting to prevent flooding during winter.

Square pad footings should be reinforced with a minimum of #4 bars, spaced 18 inches on-center each-way, and continuous footings should be reinforced with at least two #4 bars in the footing bottom in the longitudinal direction and two more in the stem wall unless determined otherwise by the Structural Engineer.

Footing excavation pits should be backfilled to grade with clay similar to the native soils. Backfill soil should be moisture–conditioned (moistened or dried as necessary) and placed as engineered fill.

5.2.2 Resistance to Lateral Loads

To ensure lateral resistance, footing concrete should cast against the ground. Alternately, the footings can be cast against forms and then the excavation backfilled with engineered fill of native-type soils.

Passive resistance, where there is at least 10 feet of level or up-sloping soil adjacent to the footing, may be computed as an equivalent fluid having a density of 210 pcf. An allowable coefficient of friction between fill soils and concrete of 0.25 may be used with dead load forces. If Caltrans AB is placed at least six inches thick, a coefficient of friction of 0.35 may be used. When combining passive pressure and friction resistance, the passive pressure component should be reduced by one third.

5.2.3 Seismic Design

The 1997 UBC Chapter IV, Earthquake Design, requires that structures be designed using certain earthquake design criteria. The criteria are based in part on the seismic zone, soil profile, and the proximity of the site to active seismic sources (faults). During an earthquake event, structures located very close to active faults can be subjected to near source energy motions that may be damaging to structures if the effects of these energy motions are not considered in the structural design. Seismic design factors "Near Source Factors" have been included in Section 4 above.

5.3 SLABS-ON-GRADE AND EXTERIOR FLATWORK

Concrete slabs should be no less than four inches thick and reinforced using #4 or larger bars, centrally placed. The reinforcement should be detailed to limit cracking to the degree desired. Fill beneath concrete slabs-on-grade should be uniform in consistency and density. All slabs on grade should be founded on at least four inches of AB, placed as engineered fill compacted to 90 percent of maximum dry density, and on a prepared subgrade of native soils as described below.

For interior slabs that will have floor finishes, provide a minimum of six inches of free draining material as a capillary break in lieu of the AB. Top this with a flexible moisture barrier such as 'visqueen' at least 15-mils thick, a layer of curing sand no more than two inches thick, and then the slab.

For design of concrete slabs on grade in accordance with the 'Beam on Elastic Foundation' method and a subgrade prepared as described, we recommend using a modulus of subgrade reaction of 100 pounds per cubic inch (pci).

Exterior concrete slabs for walkways and driveways should also be placed on four inches of compacted AB over a prepared subgrade of native soil or engineered fill. Slabs should be no less than four inches thick. Paving slabs should have control joints (saw cuts or cold joints) placed at a maximum of 10-foot intervals, in each direction, without "T-joints". As the local soils have a moderate expansion potential, we recommend that they be poured independent of footings and should not be doweled or pinned to them. Slabs-on-grade should be isolated from column foundations and walls by placing a flexible joint material that will permit differential movement.

5.4 RETAINING WALLS

Active earth pressure on retaining walls that are not restrained by corners and can rotate 0.2 percent of their height, and if backfilled with granular materials, may be designed using an active earth pressure equivalent to 33 pcf-efp and 45 pcf for sloping (rising) backfill, not steeper than 2:1 (horizontal: vertical). Such granular fill should have a minimum sand equivalent of 35 (S.E. $\geq$ 35), except for the top two feet, which should be of clayey soils similar to the native site soils.

Walls not free to rotate, which typically include basements, driveway ramps, and truck dock walls may be backfilled with clays and/or silts similar to the native site soils. Such walls should be designed using an at-rest earth pressure, which may be computed as an equivalent fluid having a density of 70 pcf. For sloping (rising) backfill, not steeper than 2:1 (horizontal: vertical) the value should be increased to 90 pcf-efp. Alternately, the walls can be backfilled with granular fill to within two feet of the ground surface. Such granular fill, with a minimum sand equivalent of 35 (S.E. $\geq$ 35), may be designed using an at rest pressure of 55 pcf-efp. The top two feet of backfill should be a clayey material similar to the native soils, with its surface sloped to direct surface water away from the wall. All backfill should be mechanically compacted as engineered fill as discussed above in Section 5.1, Site Preparation.

All retaining walls should also be designed to resist any transient or permanent surcharge loads in addition to the active or at rest pressure. If "ride-on" compaction equipment is to be operated within five feet of retaining walls, an efp of at least 100 pcf should be used in design.

The retaining walls should be backdrained. Provide a vertical drainage blanket against the wall, such as Miradrain® or a foot of drain rock (see below) down to foundation levels, and a pipe subdrain to gravity outlet and to a sump where there is no gravity outlet. A four-inch diameter perforated PVC pipe rated SDR 35 or better should be installed at the foot of the wall with at least a 1.5 percent slope. Four inches of Caltrans Class II permeable material (or 3/4-inch drain rock) should be placed on the bottom and 12 inches on the sides and top of the pipe for walls higher than 18 inches in order to prevent the build-up of hydrostatic pressure. If drain rock is used, it should be enclosed in a geofabric such as Mirafi 140N®, or equivalent. An engineered sump pump should be installed for the basement. It should be provided with a redundant back-up system.

5.5 ASPHALT PAVING

Planned asphalt paving will comprise minor transitions from the existing streets and parking lots to the new recreation center building and surrounding parking lots. The paving thicknesses presented below are based on our understanding of the project requirements. A sample for an R-value test was not collected during the geotechnical investigation because the site area soils are predominantly clays, and based on experience, would result in an R-value of approximately 5. A sample may be collected from site soils if our understanding of the design concept changes and it would appear that savings can be made by recalculating pavement thicknesses based on actual test results. The sampling can best be deferred until site grading operations begin and the majority of site soils are exposed. In addition, a more representative sample can be collected and the design revised at that time with essentially no impact to schedule.

We recommend that maneuvering aprons for dumpster trucks be paved with no less than six inches of reinforced concrete in lieu of asphalt paving. The subgrade should be prepared as herein and topped with no less than nine inches of compacted AB. Ensure that the apron and dumpster pad have positive drainage away from the recreation center building as described below in Section 5.6.

Based on an assumed R-value of 5, AC sections would be:

	TRAFFIC INDEX (TI)					
	4.0		**6.0**		**8.0**	
Full depth asphalt pavement	6.5		9.0		12.0	
AC on AB	2.5	3	3	5	5	6
	7.5	6	14	8	18	12.5
AC on AB on Aggregate Subbase (ASB)	2.5		3	4	5	6
	4		6	6	8	8
	4		9	6	12	10

Thicknesses are in inches
AB is Caltrans Class 2 AB (minimum R-value of 78)
ASB is Caltrans Class 3 aggregate subbase (minimum R-value of 40)

We recommend preparing the subgrade in paved areas by scarifying to an eight-inch depth, moisture conditioning, and then compacting to the requirements of engineered fill in Section 5.1.6. AB beneath paving should be compacted to 95 percent of maximum dry density per ASTM D1557. Asphalt pavements should be positively drained using a pipe subdrain. Curbing should extend at least three inches into the subgrade to provide a seal against ingress of irrigation water. Irrigation strips adjacent to paving should be separately subdrained.

5.6 DRAINAGE

Positive surface gradients should be maintained away from the planned buildings by engineered pavement, swales, piping, etc., such that water is not allowed to flow uncontained on site and is collected and conveyed into approved drainage devices. We recommend that drainage grades be no less than six inches in the first 10 feet away from the two buildings if grass surfaced and three inches in 10 feet if concrete or asphalt paved. Site drainage water should be conveyed to the streets or into approved drainage areas located remote from the buildings.

5.7 CORROSION

A suite of corrosion tests was performed on a bulk soil sample collected from the upper 10 feet of the site. Tests for resistivity, chloride content, sulfate content, pH and ORP (oxidation-reduction potential) were performed in accordance with current Caltrans procedures. Based on the test results, sulfate-resistant cement and additional reinforcing steel cover are not necessary. The test results are included in Appendix E.

6.0 ADDITIONAL SERVICES AND LIMITATIONS

The following sections summarize the additional services that are recommended for the project and the limitations of the findings presented in this report.

6.1 ADDITIONAL SERVICES

The review of plans and specifications, and field observation and testing during construction by 4LEAF are an integral part of the conclusions and recommendations made in this report. If 4LEAF is not retained for these services, the City of Brentwood will assume any responsibility for any potential claims that may arise during or after construction due to the misinterpretation of the recommendations presented in this report. The recommended tests, observations, and consultation by 4LEAF during construction include, but are not limited to:

- Reviewing plans and specifications
- Observing site grading operations, including stripping and engineered fill construction
- Observing foundation construction
- Evaluating materials for suitability and performing in-place density testing of fills, backfills, and finished subgrades.

6.2 LIMITATIONS

The services provided under this contract as described in this report include professional opinions and judgments based on the data collected. These services have been prepared according to generally accepted geotechnical engineering practices that exist in the San Francisco Bay area at the time the report was written. No warranty is expressed or implied. This report is issued with the understanding that the owner chooses the risk they wish to bear by the expenditures involved with the construction alternatives and scheduling that is chosen.

The conclusions and recommendations of this report are for the construction of the new recreation facility and concession stand buildings in Brentwood, California, as described in this report. The conclusions and recommendations in this report are invalid if:

- The proposed construction, as described, changes,
- The report is used for adjacent or other property,
- The Additional Services section of this report is not followed,
- Changes in grades occur between the issuance of this report and construction, or
- Any other change is implemented that materially alters the project from that proposed at the time this report was prepared.

The conclusions and recommendations presented in this report are based on information obtained from the following:

- Three CPT probings and two HSA borings performed for this investigation,
- The observations of our geotechnical engineer at the site during our field investigation,
- The results of the laboratory tests, and
- Review of previous drilling investigations conducted within the site vicinity.

The recommendations provided in this report are based on the assumption that an adequate program of tests and observations will be conducted by our firm during the construction phase to evaluate compliance with our recommendations. If we are not retained for these services, 4LEAF cannot assume any responsibility for any potential claims that may arise during or after construction as a result of misuse or misinterpretation of 4LEAF's report by others. Furthermore, 4LEAF will cease to be the Geotechnical-Engineer-of-Record at the time another consultant is retained for follow-up service to this report.

The CPT probings and HSA boring logs do not provide a warranty as to the conditions that may exist at the entire site. Soil borings were not advanced in the central portion of the site due to existing buildings. The extent and nature of subsurface soil and groundwater variations may not become evident until construction begins. It is possible that variations in soil conditions between borings could exist beyond or beyond the points of exploration or that groundwater elevations may change, both of which may require additional studies, consultation, and possible design revisions. If conditions are encountered in the field during construction which differ from those described in this report, our firm should be contacted immediately to provide any necessary revisions to these recommendations.

It is the client's responsibility to ensure that all parties to the project including the designer, contractor, subcontractors, etc., are made aware of this report in its entirety, including the Additional Services and Limitations sections.

This report may be used only by client and only for the purposes stated, within a reasonable time from its issuance. Land use, site conditions (both on-site and off-site) or other factors may change over time, and additional work may be required with the passage of time. Any party other than the client who wishes to use this report shall notify 4LEAF in writing. Based on the intended use of this report, 4LEAF may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the client or anyone else will release 4LEAF from any liability resulting from the use of this report by any unauthorized party.

7.0 REFERENCES

Association of Bay Area Governments (ABAG),Oakland, CA, 1998: On Shaky Ground-Supplement, September 1998, 28 pages. Includes ground shaking intensity maps. A supplement to The San Francisco Bay Area - On Shaky Ground, by ABAG, April 1995, 56 pages.

Biggar, N.E. and I.G. Wong, 1992: 'Seismic Hazard Evaluation of the Vaqueros Faults in Eastern Contra Costa County, California' in Borchardt et al, GSP113, p365ff.

Borchardt, Glenn, Chief Editor, S.E. Hirschfeld, J.E. Lienkamper, P. McClellan, P.L. Williams and I.G. Wong, editors, Proceedings of the Second Conference on Earthquake Hazards in the Eastern San Francisco Bay Area, California Department of Conservation, Division of Mines and Geology, Special Publication 113

CDMG, Various 7.5' quadrangle maps of the Alquist-Priolo Seismic Safety Hazard Zones.

CDMG / USGS Probabilistic Seismic-Hazard Assessment for the State of California, CDMG Open-file report 96-08.

CGS, 2004a: California Geological Survey Probabilistic Seismic Hazards Mapping: Alquist Priolo Earthquake Zones Page, May 2004.

CGS, 2004b: California Geological Survey Probabilistic Seismic Hazards Mapping: Ground Motion Page 'Ground Motions for User Selected Site', May 2004.

Carey, Debra, 1992: 'Seismic Risk Assessment in the Brentwood Area, Eastern Contra Costa County,' in Borchardt et al, GSP113, p549ff.

Contra Costa County Flood Control District, 1977, County Hydrology Standards.

Contra Costa County, 1991, *General Plan,* Safety Element, Section 10.

Crane, R.C, 1995: 'Geology of the Mount Diablo Hills and East Bay Hills' in Recent Geologic Studies of the San Francisco Bay Area. SEPM, Pacific Region, Vol. 76, p87-114. Includes geologic map of USGS Brentwood quadrangle (1:24,000).

4LEAF, Inc., 2004. Geotechnical Investigation Report for the New Brentwood City Hall Building, 708 Third Street, Brentwood, CA., May 5.

4LEAF, Inc., 2004. Foundation Design Recommendations Letter for the New Brentwood Recreation Facility and Concession Stand Buildings, Brentwood, CA., August 4.

Galehouse, J.S. 1992: 'Creep-Rates And Creep-Characteristics Of Eastern San Francisco Bay Area Faults:1979-1992' in Borchardt et al, GSP113, p45-53.

Gibbs, H. J. and Bara, J. P. 1962 "Predicting Surface Subsidence form Basic Soil Tests", ASTM STP 322, pp 231-247. Also numerous subsequent US Bureau of Reclamation research publications by Gibbs, Bara, W.G. Holtz, J.W. Hilf, N.P. Prokopovich, and others.

Gregg Drilling, 2004, CPT Site Investigation Results Report, Brentwood Recreation Facility, Brentwood, CA., July 9.

Hart 1988, Fault Hazard Zones in California, California Division of Mines & Geology Special Publication 42.

Helley, E.J., and Graymer, R.W., 1997, Quaternary geology of Contra Costa County and surrounding parts of Alameda, Marin, Sonoma, Sacramento, and San Joaquin Counties, California: A digital database: U.S. Geological Survey Open-File Report 97-98, includes plotfiles for 2 sheets, scale 1:100,000, database description pamphlet, 13 pp, geologic description and interpretation pamphlet, 9 pp.

Nilson, et al. 1975: Preliminary Photo-Interpretation Map of Landslide and Other Surficial Deposits of 56 USGS Quadrangles at a scale of 1:24,00. U.S.G.S OFR 75-277.

Provenzano, Joseph, M., P.E., 1999. Report of a Geotechnical Investigation for the Proposed Brentwood Municipal Swimming Pool, Brentwood, CA., March 23.

Pyke, Robert, P.E., 2003. Discussion of Youd and Idris 2001 in J. Geotechnical and GeoEnvironmental Engineering, pp 283 by The GeoInstitute of the American Society of Civil Engineers, March 2003.

Wills, Christopher J. 1992: 'The Elusive Antioch Fault', in Borchardt et al, GSP113, p3265ff.

Welch, L E., 1977: Soil Survey of Contra Costa County, California, US Dept. of Agriculture, Soil Conservation Service in cooperation with University of California Agricultural Experiment Station, issued September 1977.

Youd , T.L, and Hoose, S.N.; 1978: Historic Ground Failures In Northern California Associated With Earthquakes. U.S. Geological Survey Professional Paper 993

Youd, T.L, Idriss, I.M. et al, 2001: Liquefaction Resistance of Soils: Summary Report from the 1966 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils; in J. Geotechnical and GeoEnvironmental Engineering, pp 817-833 by The GeoInstitute of the American Society of Civil Engineers, October 2001.

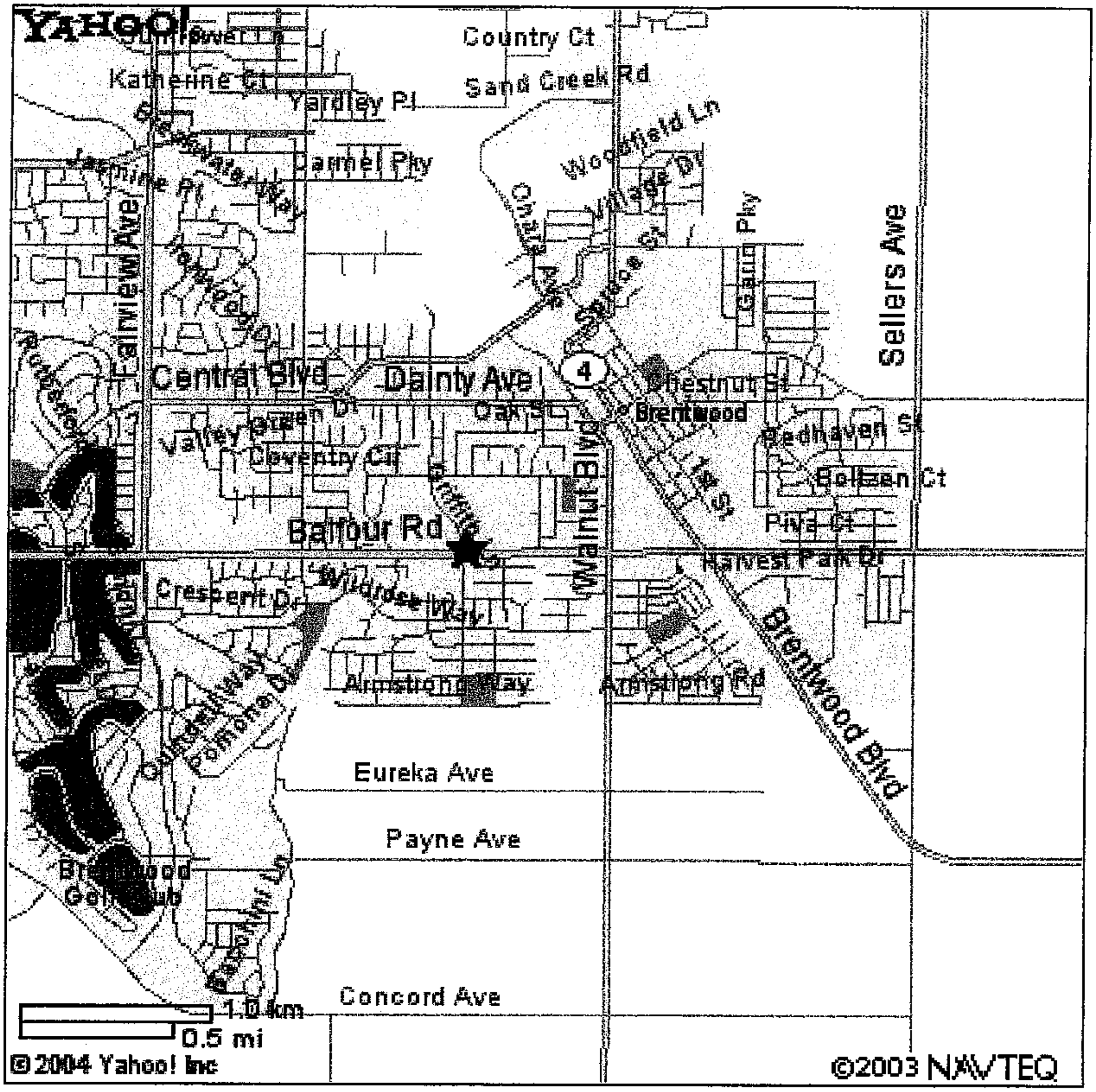


Figure 1. Site Vicinity Map.

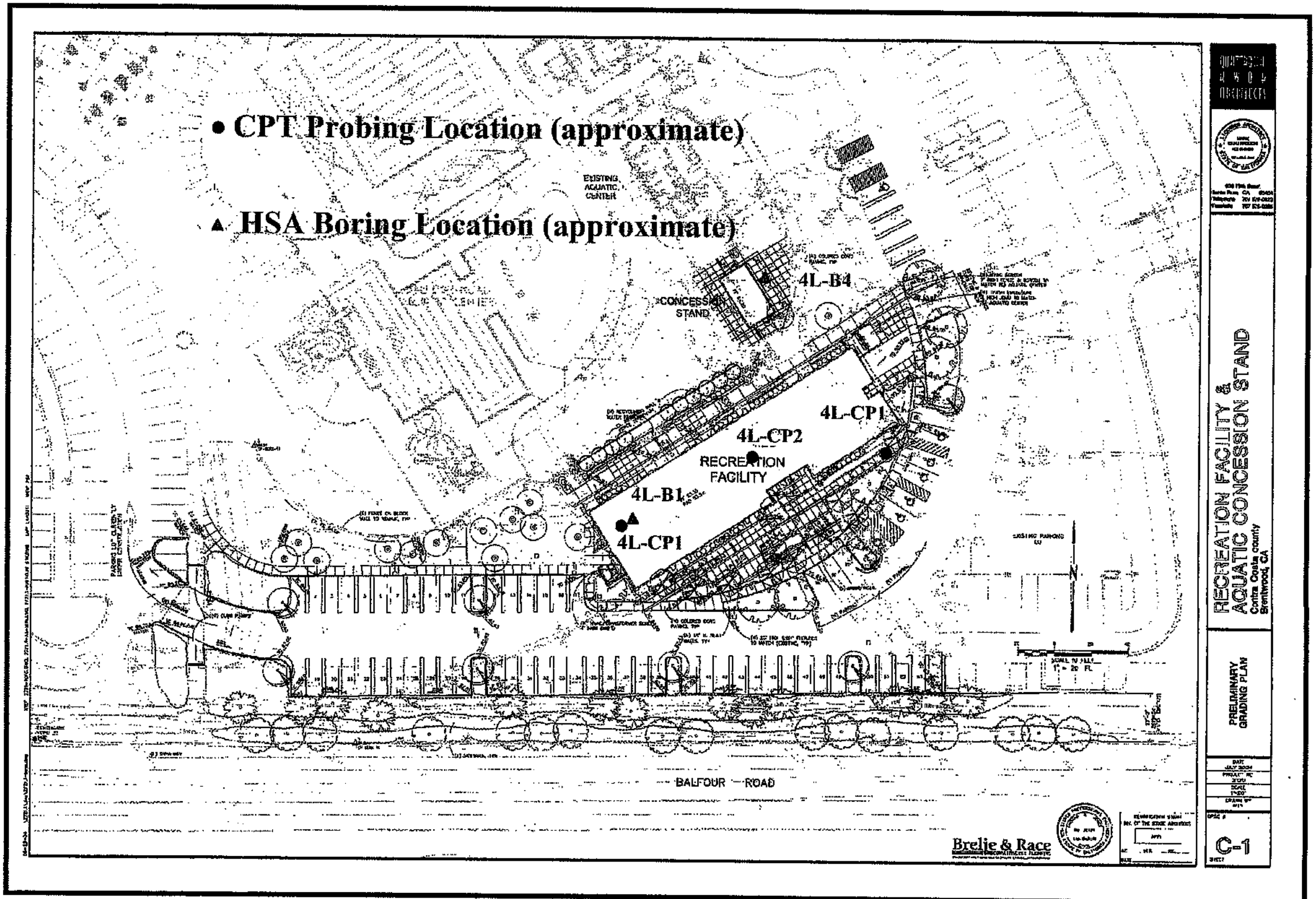


Figure 2. Site Plan.

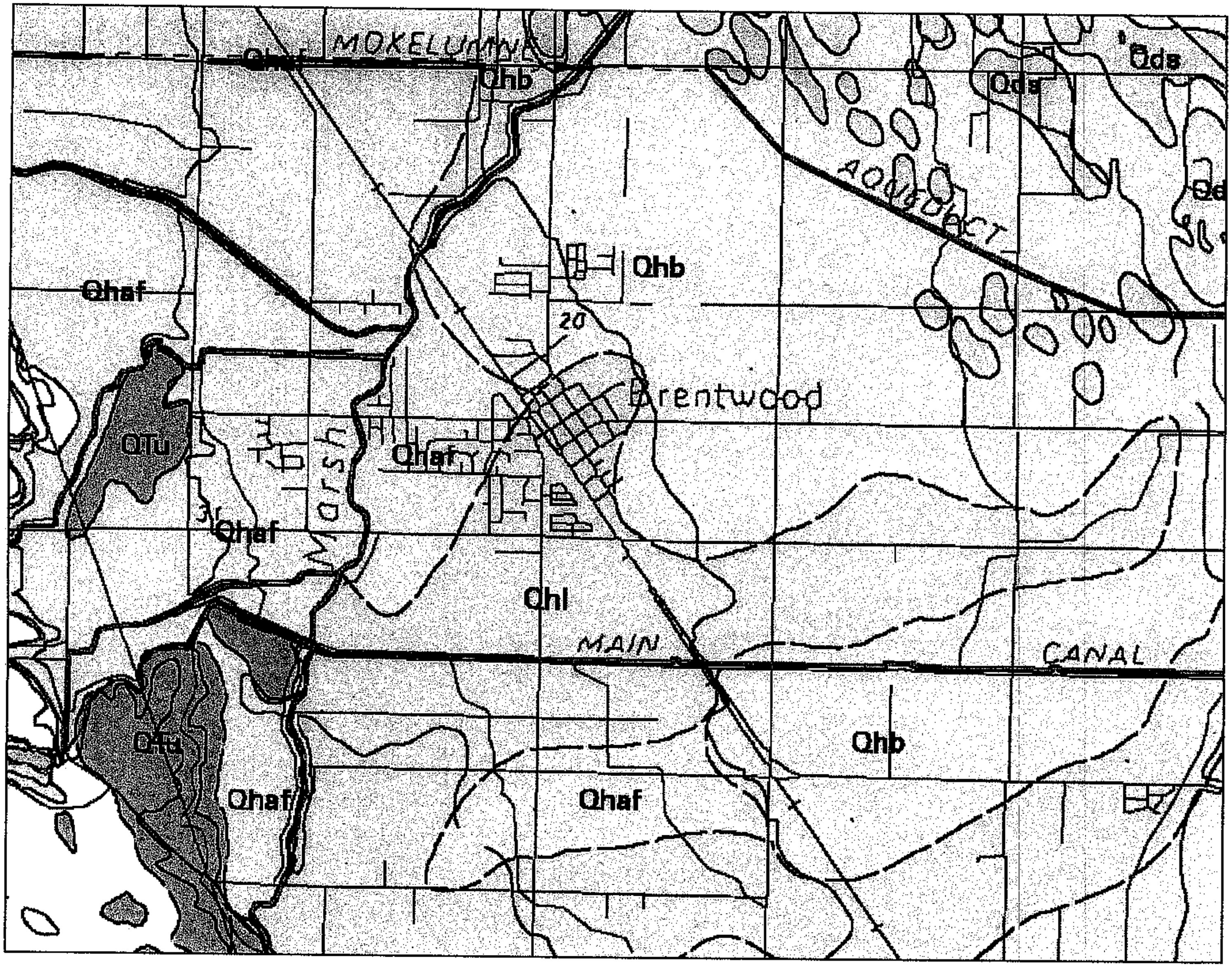
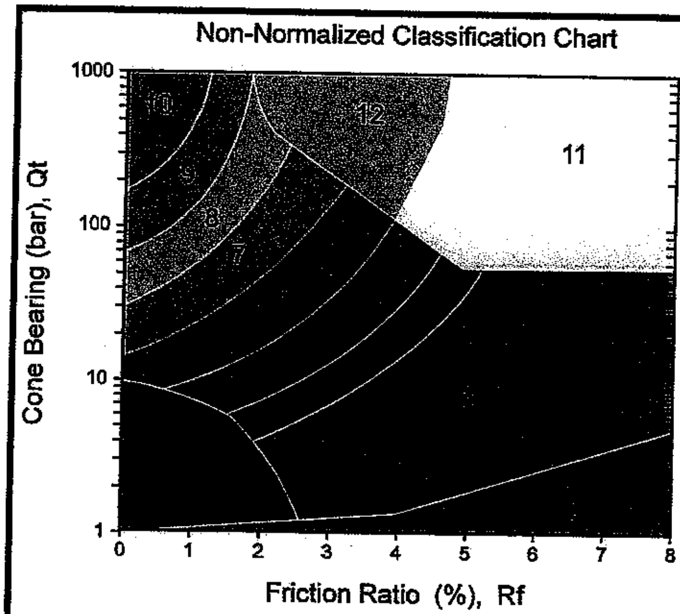
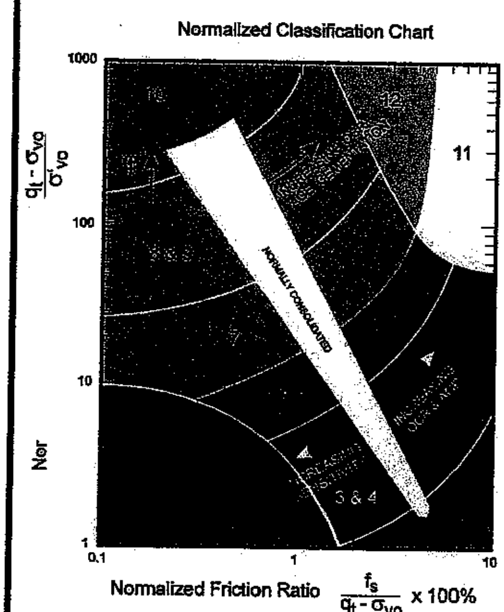


Figure 3. Regional geology.

CPT Interpretations





Zone	Q_t / N	Soil Behaviour Type
1	2	sensitive fine grained
2	1	organic material
3	1	clay
4	1.5	silty clay to clay
5	2	clayey silt to silty clay
6	2.5	sandy silt to clayey silt
7	3	
8	4	and to silty sand
9	5	sand
10	6	gravelly sand to sand
11	1	very stiff fine grained *
12	2	sand to clayey sand *

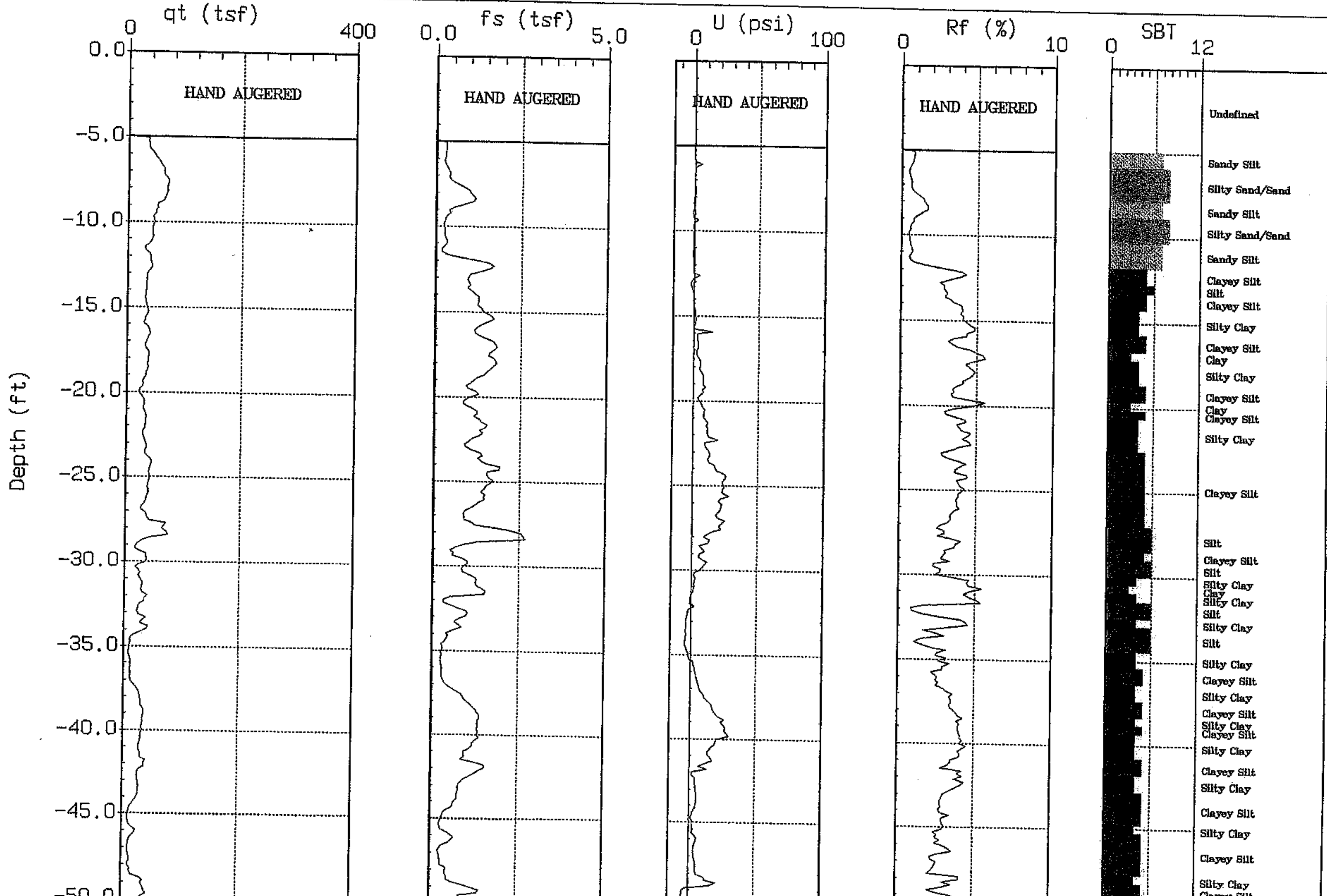
Figure 1 Non-Normalized and Normalized Soil Behavior Type Classification Charts

GREGG

4–LEAF

Site: BRENTWOOD REC.
Location: 4L-CP1

Engineer: J. SUTTON
Date: 07:02:04 11:15



Max. Depth: 50.03 (ft)
Depth Inc.: 0.164 (ft)

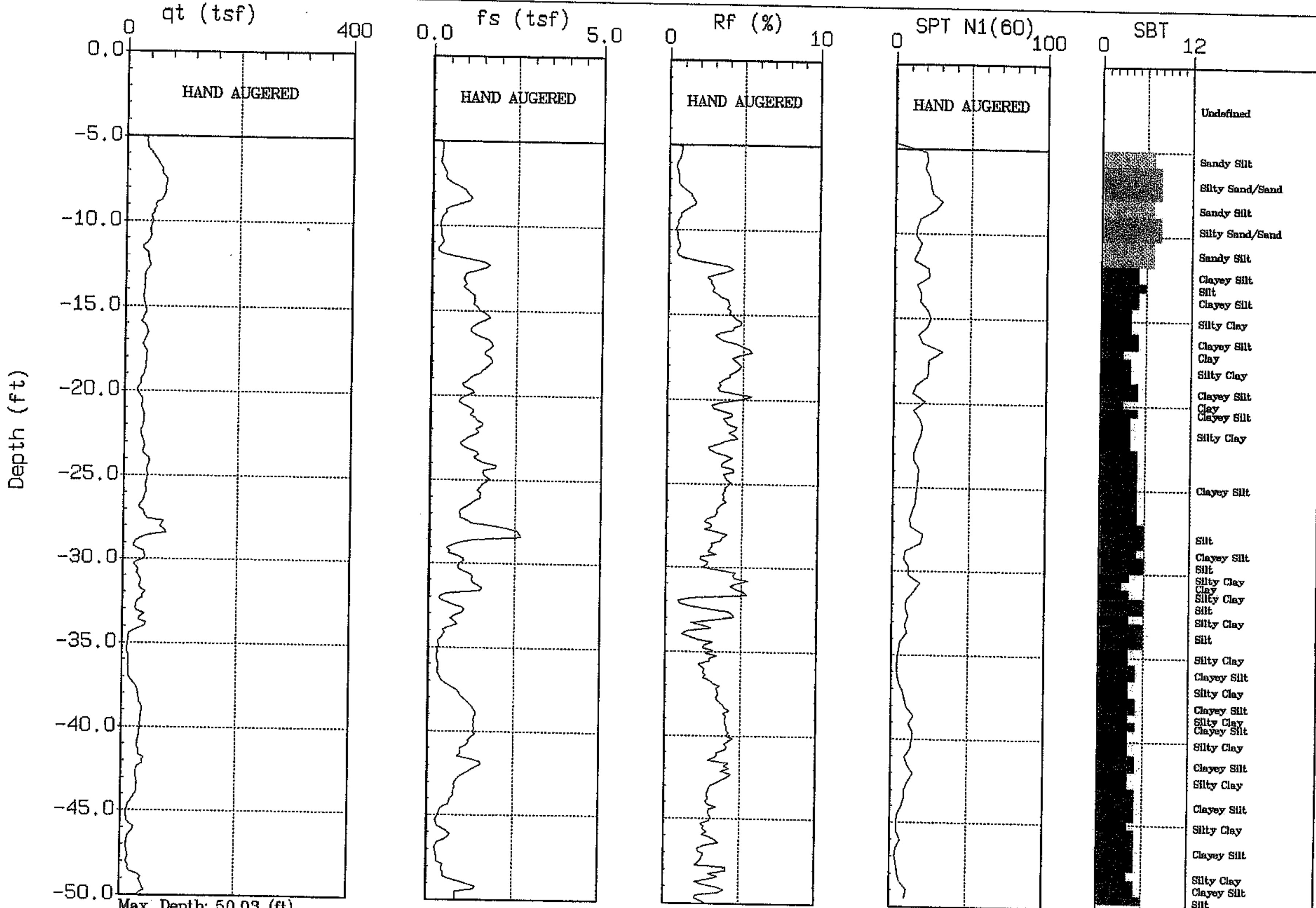
SBT: Soil Behavior Type (Robertson 1990)

GREGG

4-LEAF

Site: BRENTWOOD REC.
Location: 4L-CP1

Engineer: J. SUTTON
Date: 07:02:04 11:15



Max. Depth: 50.03 (ft)
Depth Inc.: 0.164 (ft)

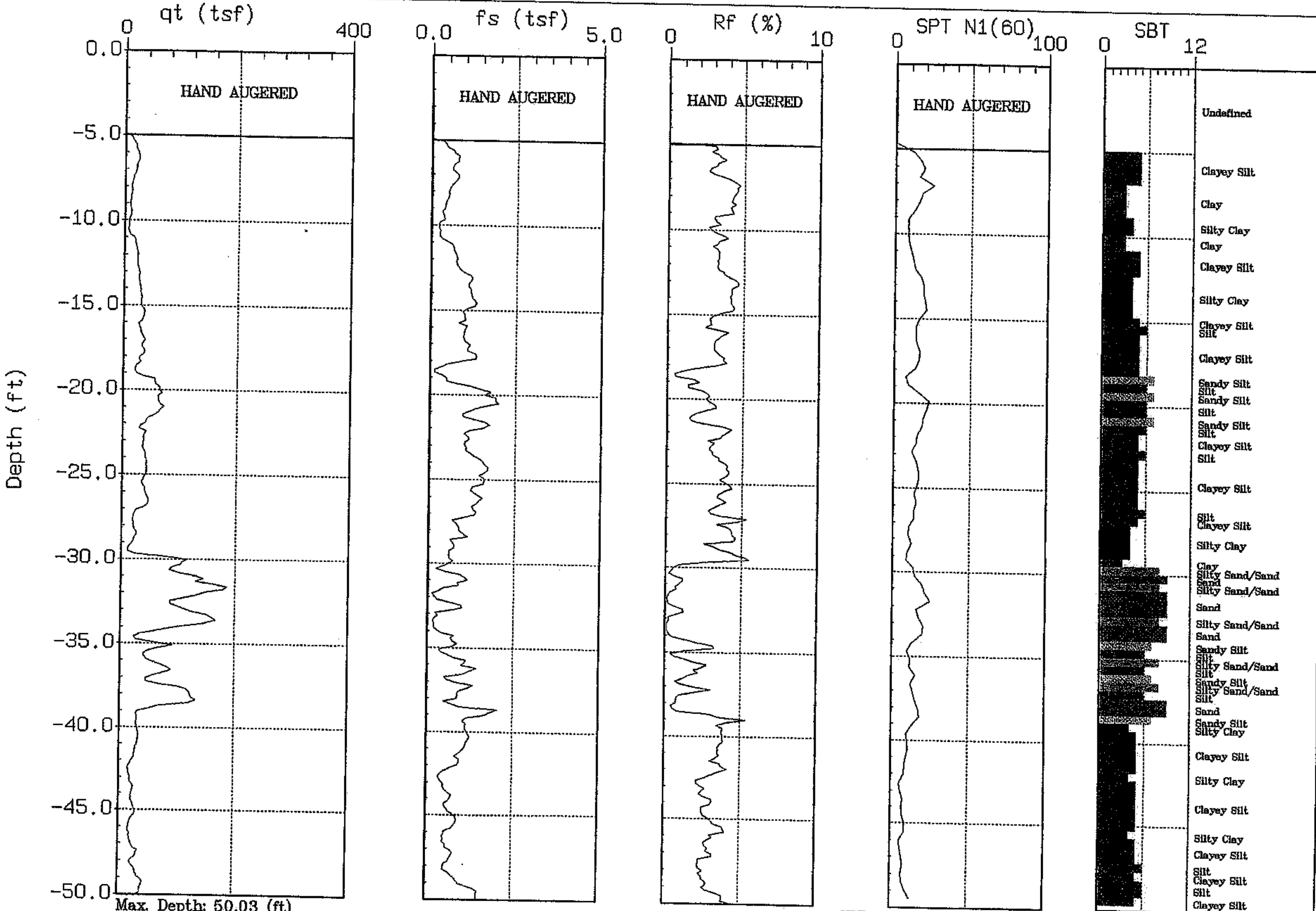
SBT: Soil Behavior Type (Robertson 1990)

GREGG

4-LEAF

Site: BRENTWOOD REC.
Location: 4L-CP2

Engineer: J. SUTTON
Date: 07:02:04 10:08



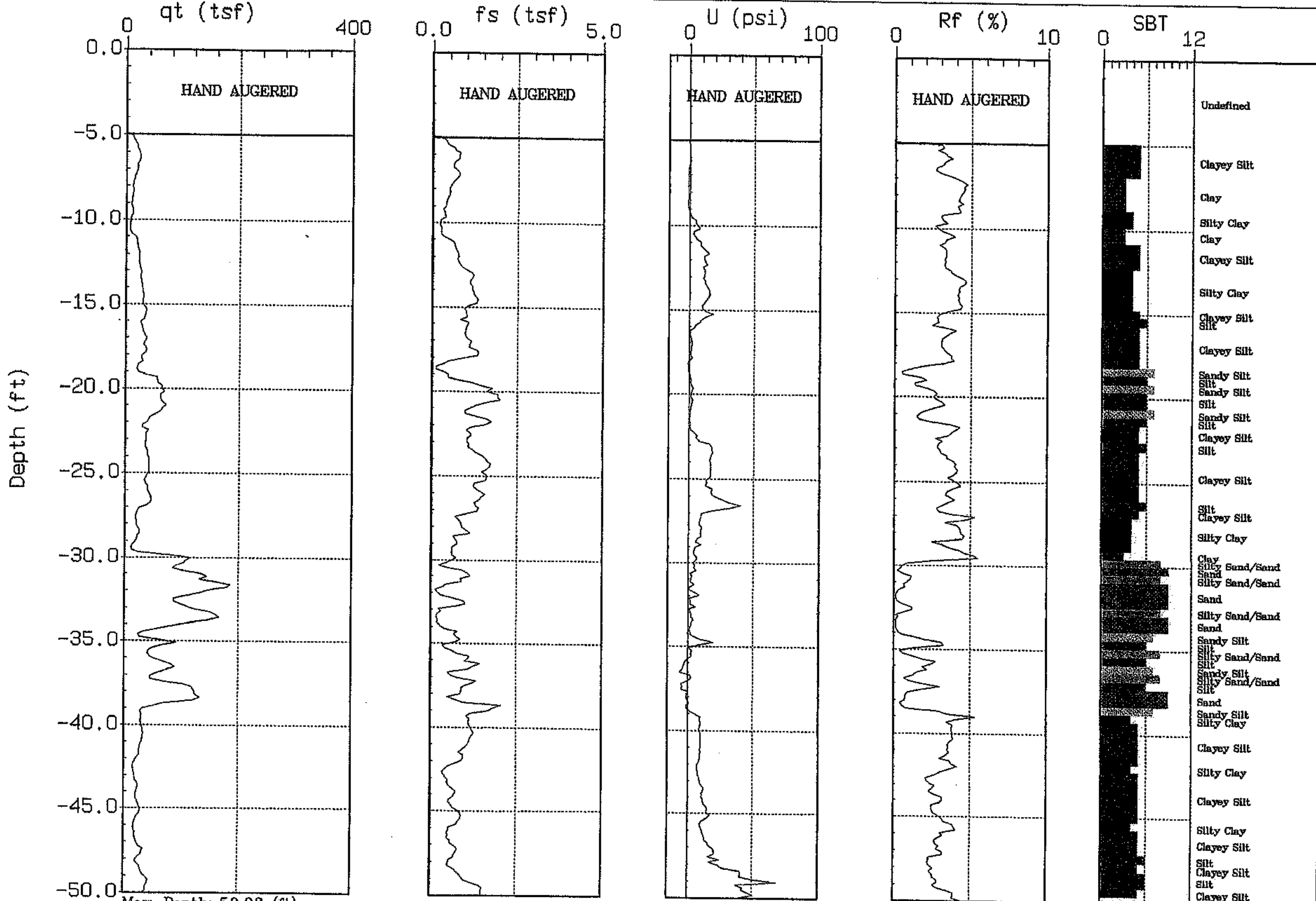
Max. Depth: 50.03 (ft)
Depth Inc.: 0.164 (ft)

SBT: Soil Behavior Type (Robertson 1990)

4-LEAF

Site: BRENTWOOD REC.
Location: 4L-CP2

Engineer: J. SUTTON
Date: 07:02:04 10:08



Max. Depth: 50.03 (ft)
Depth Inc.: 0.164 (ft)

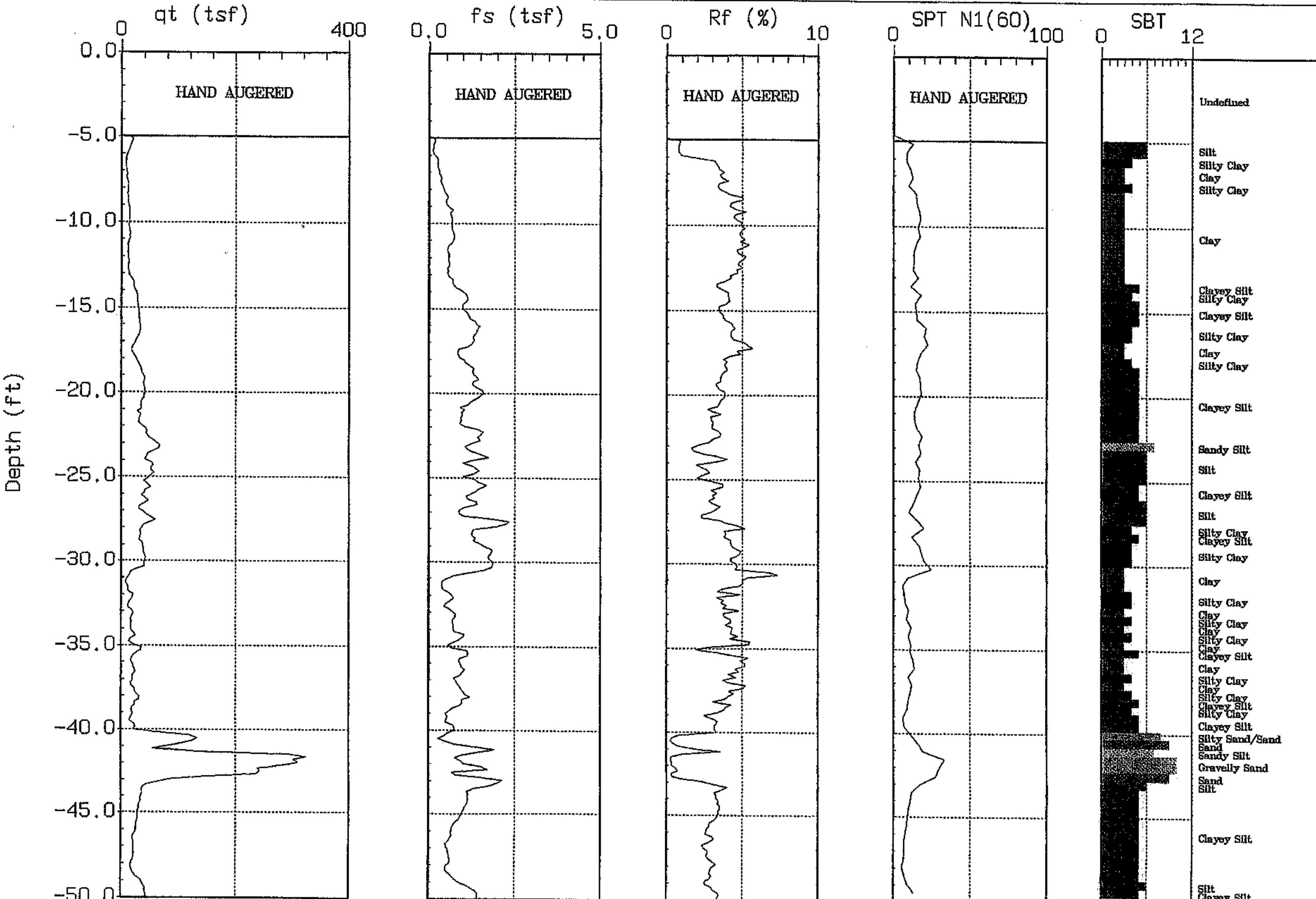
SBT: Soil Behavior Type (Robertson 1990)

GREGG

4-LEAF

Site: BRENTWOOD REC.
Location: 4L-CP3

Engineer: J. SUTTON
Date: 07:02:04 08:40



Max. Depth: 50.03 (ft)
Depth Inc.: 0.164 (ft)

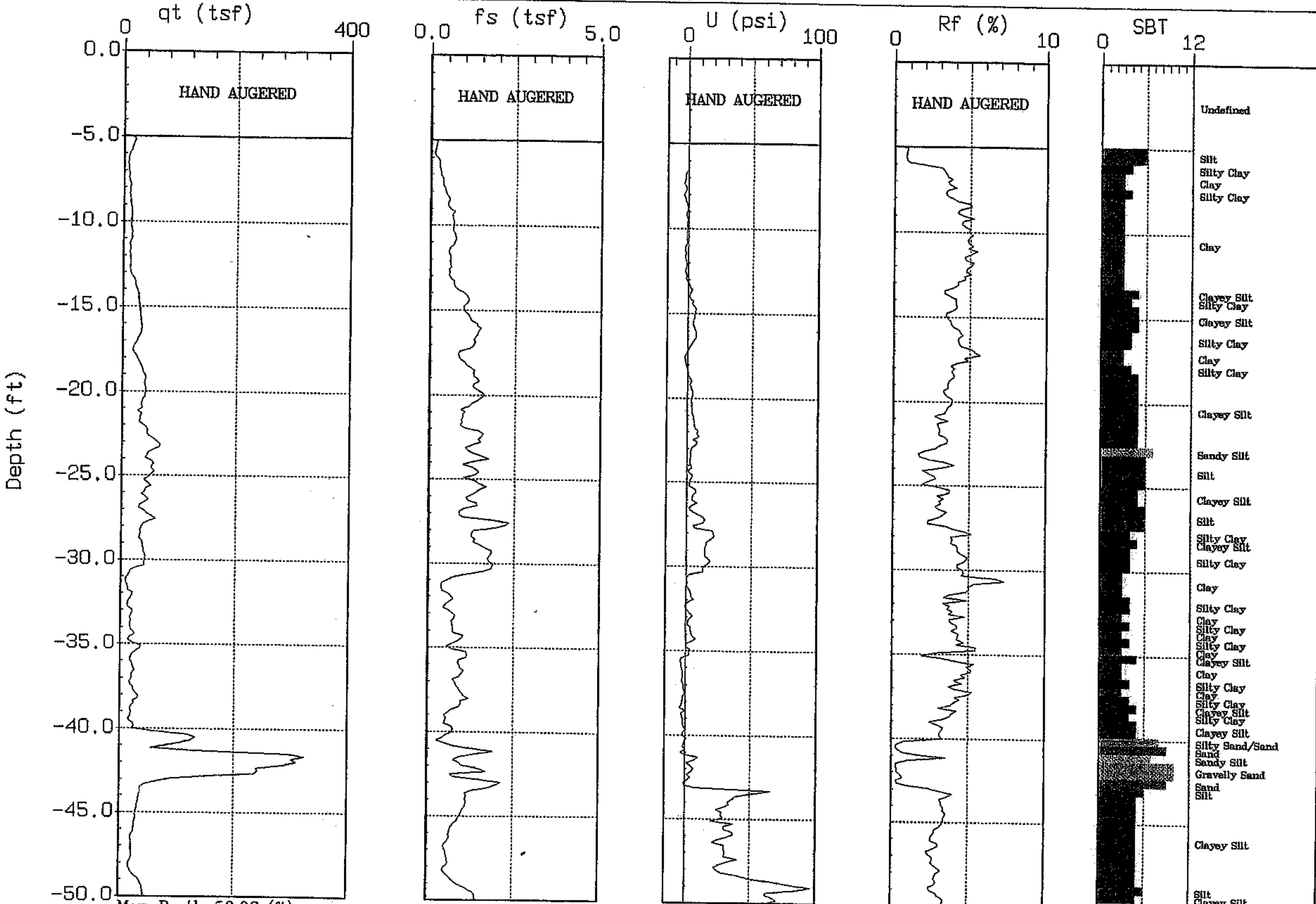
SBT: Soil Behavior Type (Robertson 1990)

GREGG

4–LEAF

Site: BRENTWOOD REC.
Location: 4L-CP3

Engineer: J. SUTTON
Date: 07:02:04 08:40



Max. Depth: 50.03 (ft)
Depth Inc.: 0.164 (ft)

SBT: Soil Behavior Type (Robertson 1990)

Cone Penetration Test Sounding Summary

-Table 1-

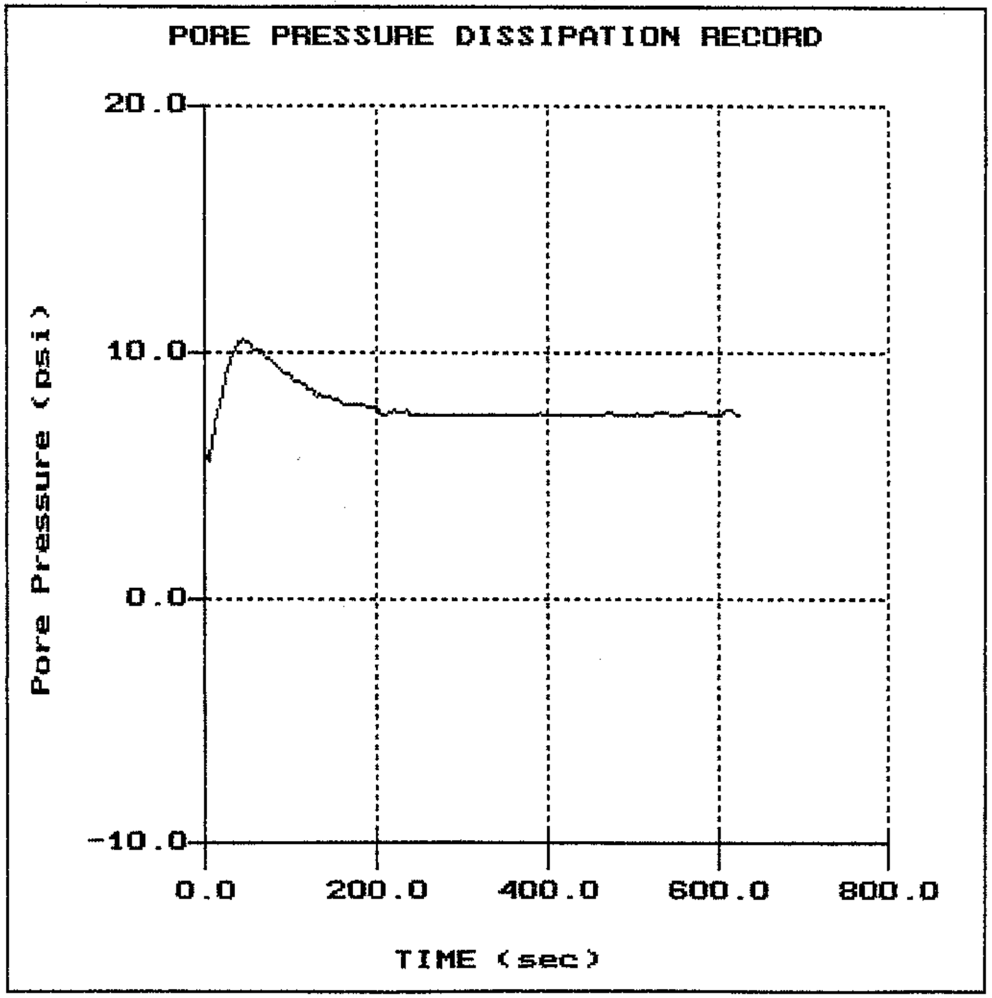
CPT Sounding Identification	Date	Termination Depth (Feet)	Depths of Groundwater Samples	Depths of Soil Samples	Depth of Pore Pressure Dissipation Tests
4L-CP1	7/2	50	-	-	48
4L-CP2	7/2	50	-	-	-
4L-CP3	7/2	50	-	-	41

4-LEAF

Site: BRENTWOOD REC.
Location: 4L-CP1

Engineer: J. SUTTON
Date:07:02:04 11:15

File: 228C01.PPC
Depth (m): 14.65
(ft): 48.06
Duration : 625.0s
U-min: 5.59 5.0s
U-max: 10.48 45.0s

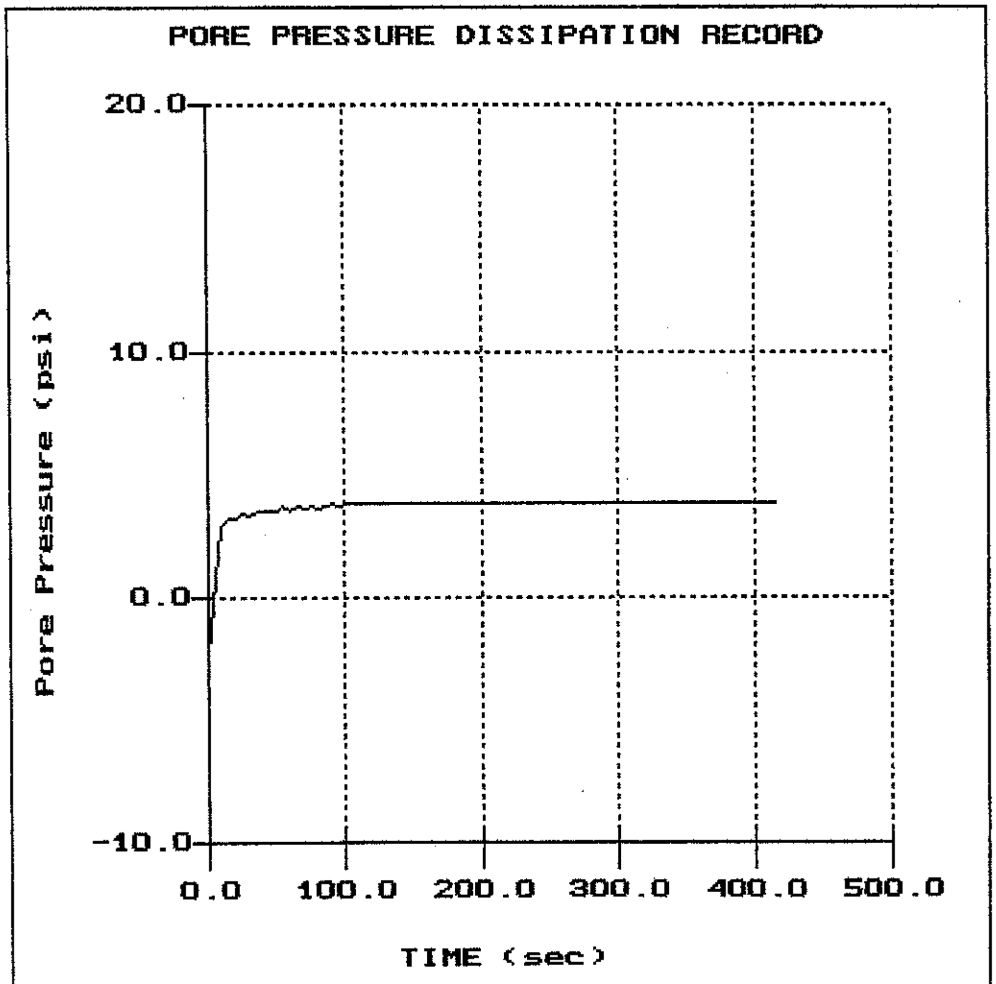


4—LEAF

Site: BRENTWOOD REC.
Location: 4L-CP3

Engineer: J. SUTTON
Date:07:02:04 08:40

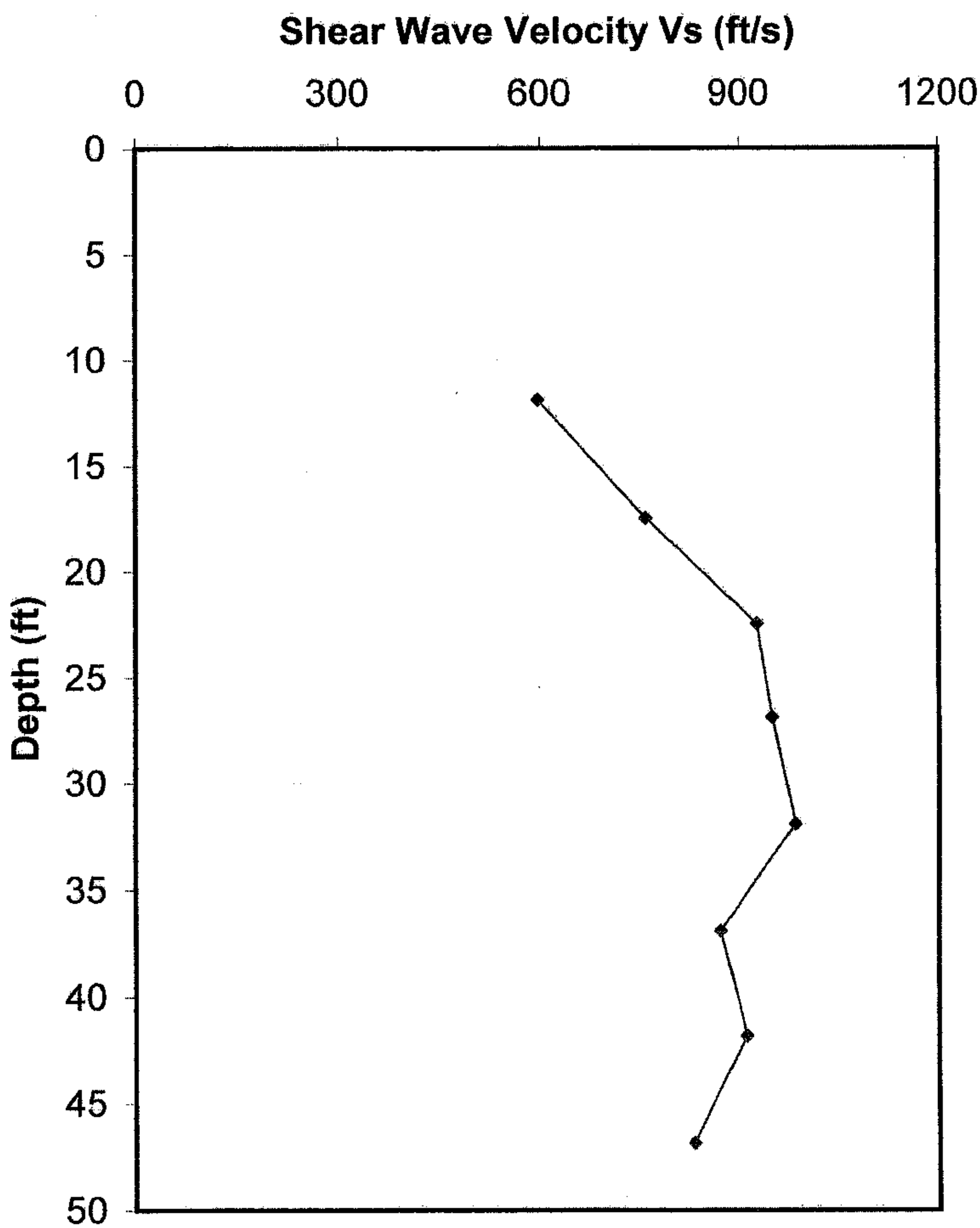
File: 228C03.PPC
Depth (m): 12.40
(ft): 40.68
Duration : 415.0s
U-min: -2.32 0.0s
U-max: 3.91 415.0s



Shear Wave Velocity Profile

GREGG

Client: 04-228ma - 4-Leaf
Site: Brentwood Recreational Facility
Location: SCPT-02
Sounding Date: July 2, 2004



GREGG

GIS Project No:	04-228MA
Client:	4-Leaf
Site:	Brentwood Recreational Facility
Location	SCPT-02
Sounding Date:	July 2, 2004

Shear Wave Velocity Calculations

Geophone Offset	(feet)	0.66
Source Offset	(feet)	1.50

Test Depth (feet)	Geophone Depth (feet)	Ray Path (feet)	Incremental Distance (feet)	Time Interval (ms)	Interval Velocity (ft/s)	Interval Depth (feet)
10	9.34	9.46				
15.09	14.43	14.51	5.05	8.46	597	11.89
21.16	20.50	20.56	6.05	7.99	757	17.47
25.09	24.43	24.48	3.92	4.23	927	22.47
30.01	29.35	29.39	4.91	5.17	950	26.89
35.10	34.44	34.48	5.08	5.17	983	31.90
40.02	39.36	39.39	4.92	5.64	872	36.90
44.94	44.28	44.31	4.92	5.40	911	41.82
50.02	49.36	49.39	5.08	6.11	831	46.82

Interval Depth (feet)	Vs (ft/s)	Vs (m/s)	Interval Depth (m)
11.89	597	182	3.62
17.47	757	231	5.32
22.47	927	283	6.85
26.89	950	290	8.20
31.90	983	300	9.72
36.90	872	266	11.25
41.82	911	278	12.75
46.82	831	253	14.27

4LEAF, INC.

BORING LOG

BORING No.: 4L-B1
Sheet 1 of 1

Project No.	J0148	Drilling Company	Gregg Drilling
Date Drilled	July 12, 2004	Drill Rig Model	B61 — Driller: Jason
Client	City of Brentwood Engineering Department	Drilling Method	Hollow Stem Auger
Site Address	195 Griffith Lane, Brentwood, CA	Borehole Diameter	8 inches
Boring Location	Adjacent to CPT boring 4L-CP1	Sampling Method	Modified California Samplers
Surface Elevation	84 ft (approx.) — Datum: msl.	Time	
Logged By	John Sutton	Wtr Depth	32' at drilling, 30' after 1.5 hours

DEPTH FEET	SAMPLE # &TYPE	BLOWS / 6 IN.	N	SYM	USCS CLASS	DESCRIPTION	Comment
1						Fill – silty clay, light brown, dry.	
						Composite sample collected from 0 – 6' at borings 4L-B1 & 4L-B4 for M-D Curve.	
	1-1	4,6,7	13		CL	Fill - sandy clay, porous, brown, dry.	MD, Att. Limits
	C25					w = 7%, DD = 85 pcf.	Bulk M-D Curve
5	1-2	5,6,8	14		ML	Brown silt with sand, slightly plastic. % minus #200 = 83%, w = 11%, DD = 83 pcf.	Collapse test,
	1-3	13,17,21	38		ML	Brown sandy silt, slightly plastic, dry to moist.	% minus #200
						w = 13%	M
	1-4	11,17,12	29		CL	Brown sandy clay	MD
						w = 15%, DD = 104 pcf.	
10							
	1-5	14,16,23	39		CL	Brown sandy clay, moist to very moist.	MD
	C25					w = 19%, DD = 109 pcf.	
15							
	1-6	17,16,20	36		CL	Brown sandy clay, moist.	MD
20						w = 16%, DD = 114 pcf.	
	1-7	8,15,15	30		CL	Brown sandy clay, very moist	MD
	C25					w = 16%, DD = 113 pcf.	
25							
	1-8A	8,8,7	15		SC	Brown clayey sand (sample 1-8A), Brown sandy silt (sample 1-8B).	M
30	1-8B					Sample 1-8A: w = 12%, sample 1-8B: w = 22%	▼
	SPT					Groundwater tagged at 30 feet below ground surface.	
							▽
	1-9A	3,4,4	8		ML	Brown silty sand (sample 1-9A), Brown sandy silt (sample 1-9B).	M
35	1-9B					33.5 – 34 ft: w = 25%, 34.5 – 35 ft: w = 32%.	
	SPT						
	1-10	8,11,12	23		CL	Brown clay, moderately plastic.	MD
40	C25					w = 24%, 103 pcf.	
						Total depth of boring = 40 feet below surface.	
45							

SAMPLER Type: SPT = 2" OD SPT; C20 = 2" ID California, C25 = 2½ " ID California, ST = Shelby, P = Pitcher
DD = dry density, pcf; w = water content %; LL= liquid limit, PI = plasticity index; UCS=Unconfined shear strength, psf.

4LEAF, INC.

BORING LOG

BORING No.: 4L-B4

Sheet 1 of 1

Project No.	J0148	Drilling Company	Gregg Drilling
Date Drilled	July 12, 2004	Drill Rig Model	B61 — Driller: Jason
Client	City of Brentwood Engineering Department	Drilling Method	Hollow Stem Auger
Site Address	195 Griffith Lane, Brentwood, CA	Borehole Diameter	8 inches
Boring Location	At proposed concession stand, outside fence	Sampling Method	Modified California Samplers, SPT
Surface Elevation	84 ft (approx.) — Datum: msl	Time	
Logged By	John Sutton	Wtr Depth	32.0 ft at drilling

DEPTH FEET	SAMPLE # &TYPE	BLOWS / 6 IN.	N	SYM	USCS CLASS	DESCRIPTION	Comment
1					CL	Lawn, top soil. Dark brown clay with sand. % minus #200 = 83%, S.G. = 2.69	Bulk M-D Curve
	Bulk					Composite sample collected from 0 – 6' at borings 4L-B1 & 4L-B4 for M-D Curve.	% minus #200, SG
	4-1	7,9,10	19		SM	Brown silty sand.	M
	C25					w = 19%, DD = 97 pcf.	
5	4-2	3,8,8	16		ML	Brown silt with sand, slightly plastic.	DS
						w = 26%, DD = 96 pcf, DS: c = 240 psf, φ = 17°	
	4-3	7,11,10	21		CL	Brown sandy lean clay, w = 21%, DD = 101 pcf, LL = 40, PL = 18, PI = 22	MD, Att. Limits
	4-4	6,8,9	17		ML	Brown sandy silt, w = 21%, DD = 101 pcf	MD
	4-5	5,6,8	14			Brown silt, w = 25%, DD = 96.5 pcf	MD, Consol.
10							
	4-6	10,14,18	32		CL	Brown sandy clay	MD
15	C25					w = 18%, DD = 107 pcf	
	4-7	11,11,13	24		CL	Brown sandy clay	MD
20						w = 19%, DD = 101 pcf	
	4-8	6,8,15	23		CL	Brown sandy clay	MD
25						w = 19%, DD = 107 pcf	
	4-9	5,6,11	17		ML	Brown sandy silt	MD
30						w = 20%, DD = 105 pcf	
							∇
	4-10	2,2,4	6		SP, SC	Brown clean to clayey sand layers	MD
35	C25					w = 24%, DD = 101 pcf	
	4-11	7,8,10	18		CL	Brown sandu clay	M
40	SPT					w = 26%	
	4-12	5,4,10	14		CL	Brown clay, w = 30%, DD = 95 pcf	MD
45	C25					Total depth of boring = 45 feet below surface.	

SAMPLER Type: SPT = 2" OD SPT; C20 = 2" ID California, C25 = 2½ " ID California, ST = Shelby, P = Pitcher
DD = dry density, pcf; w = water content %; LL= liquid limit, PI = plasticity index; DS=direct shear strength, psf.

Construction Testing Services
2142 Rheem Drive, Suite E
Pleasanton, CA 94588

Laboratory Material Compaction Test Report per ASTM D 1557

Information to be provided by Field Technician:

Job Name: Brentwood Aquatic Park
Contractor: N/A
Job Number: 2869
Material Description: Clay with minor sands, light brown
Sampling Date: 07/14/04
Material Collected by: JS
Source: 0-6' Boring # 4B-1 and 4B-4.

Information to be provided by Laboratory Engineer / Technician:

Date Received: 07/15/04
Lab. Engineer / Technician: DQ
Date Tested: 07/21/04
Method Used (A, B, or C): B
Preparation Method: Wet
Test / Login Number: 21972
Rammer Desription: Mechanical

Laboratory Determined Moist and Dry Density

Water Adjust. (gms)	Moist Density (mg / m³)	Moist Density (kN / m³)	Moist Density (lbf / ft³)	Moisture Content (%)	Dry Density (mg / m³)	Dry Density (kN / m³)	Dry Density (lbf / ft³)	Rock Corr. Moisture Content (%)	Rock Corr. Dry Den. (lbf / ft³)
150	2.17	21.26	135.36	13.58	1.91	18.72	119.18		
100	2.18	21.38	136.08	12.51	1.94	19.00	120.95		
50	2.17	21.31	135.67	11.01	1.96	19.20	122.22		
0	2.11	20.70	131.76	9.27	1.93	18.94	120.58		
-50	2.05	20.13	128.12	8.60	1.89	18.53	117.97		

As Received Water Content, % (if determined): N/A

Curve Results (adjusted for rock correction)

Mod. Opt. Moisture Content, (nearest 0.5%):
Mod. Max. Dry Density, (nearest 0.5 lbf / ft³):

Curve Results (not adjusted for rock correction)

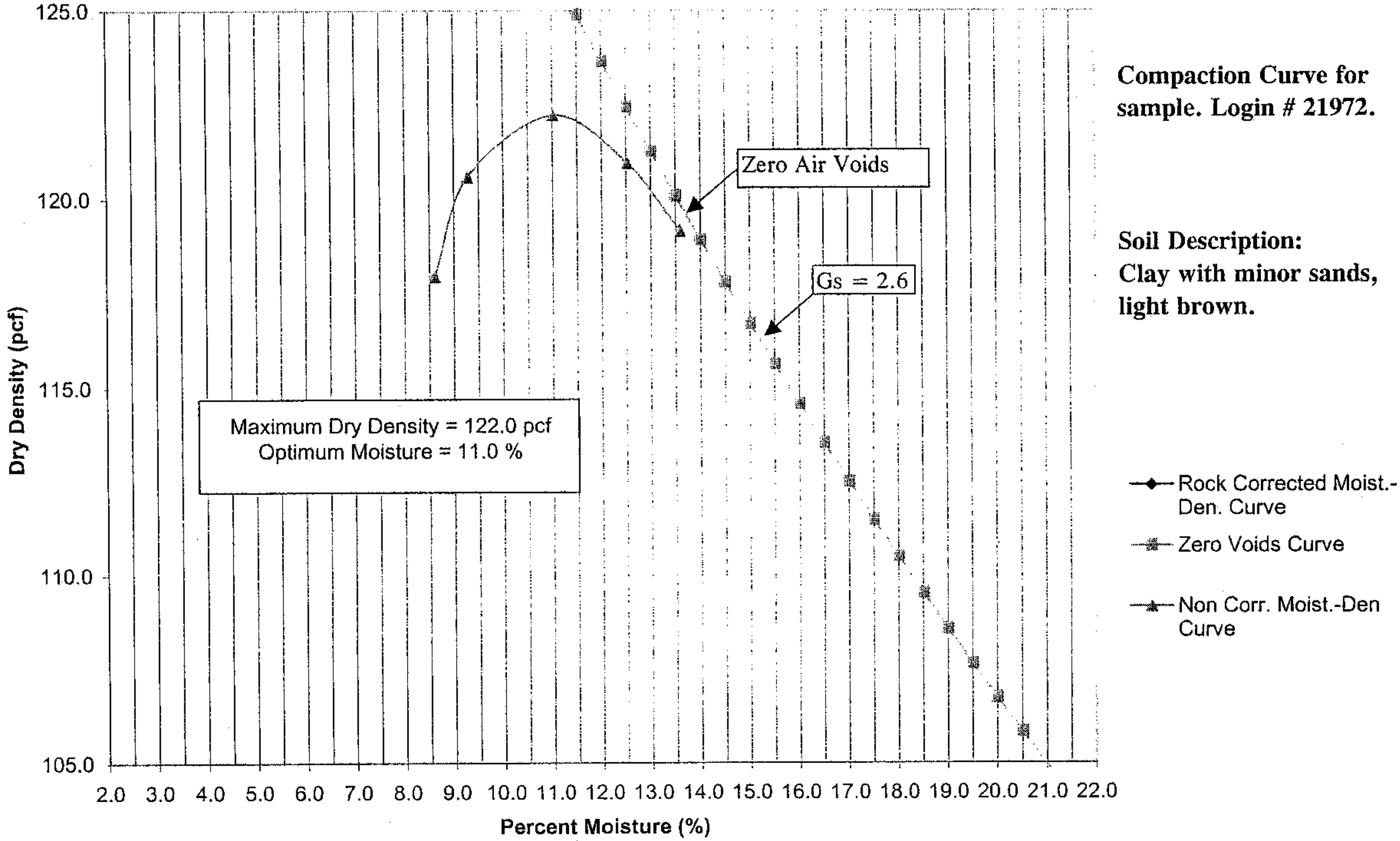
Opt. Moisture Content, (nearest 0.5%): 11.0
Max. Dry Density, (nearest 0.5 lbf / ft³): 122.0

Comments:

Reviewed by: Dexter Quidilla, Staff Engineer
7/25/04
Date

Tel: (925) 462-5151
Fax: (925) 462-5183

Moisture-Density Curve



Compaction Curve for sample. Login # 21972.

Soil Description:
Clay with minor sands, light brown.

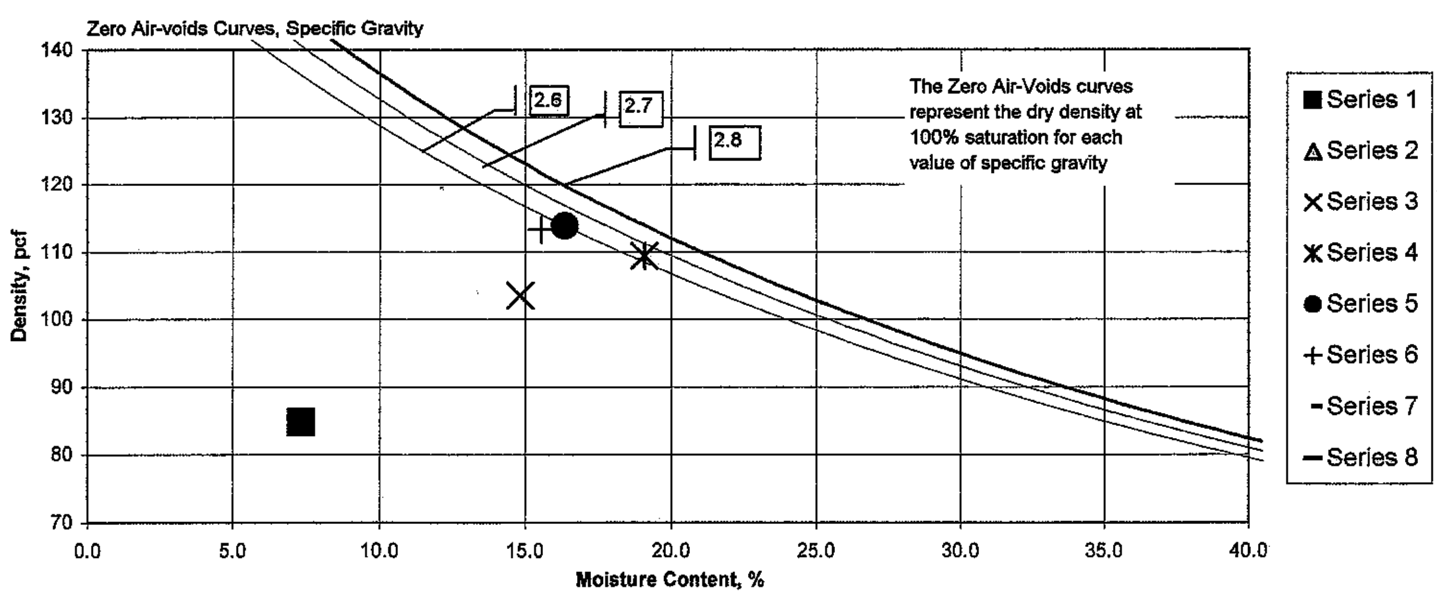
Moisture-Density-Porosity Report

Cooper Testing Labs, Inc.

ob No:	519-008a	Date:	07/22/04
ient:	4LEAF, Inc.	By:	MJ
oject:	Brentwood Rec. Ctr. - J148	Remarks:	4B1, 1-3 @ 6.5-8: Sample disturbed; M/C only.

oring:	4B1	4B1	4B1	4B1	4B1	4B1	4B1	4B1
mple:	1-1	1-3	1-4	1-5	1-6	1-7	1-8A	1-8B
epth, ft:	3.5-5	6.5-8	8-9.5	13.5-15	18.5-20	23.5-25	28.5-30	28.5-30
sual escription:	Brown Sandy Lean CLAY	Brown Sandy SILT, slightly plastic	Brown Sandy CLAY	Brown Sandy CLAY	Brown Sandy CLAY	Brown Sandy CLAY	Brown Clayey SAND	Brown Sandy SILT
tual G:								
sumed G:	2.70		2.70	2.70	2.70	2.70		
tal Vol cc	223.69		441.45	221.83	223.69	224.04		
l Solids,cc	112.42		271.04	143.82	150.94	150.42		
l Voids,cc	111.27		170.41	78.01	72.75	73.62		
oisture, %	7.3	13.4	14.8	19.1	16.4	15.6	11.9	21.8
t Unit wt, pcf	91.0		118.9	130.2	132.4	130.9		
y Unit wt, pcf	84.8		103.6	109.4	113.8	113.3		
turation, %	20.0		63.7	95.0	91.6	85.8		
rosity, %	49.7		38.6	35.2	32.5	32.9		
id Ratio	0.990		0.629	0.542	0.482	0.489		
ries	1	2	3	4	5	6	7	8

Moisture-Density

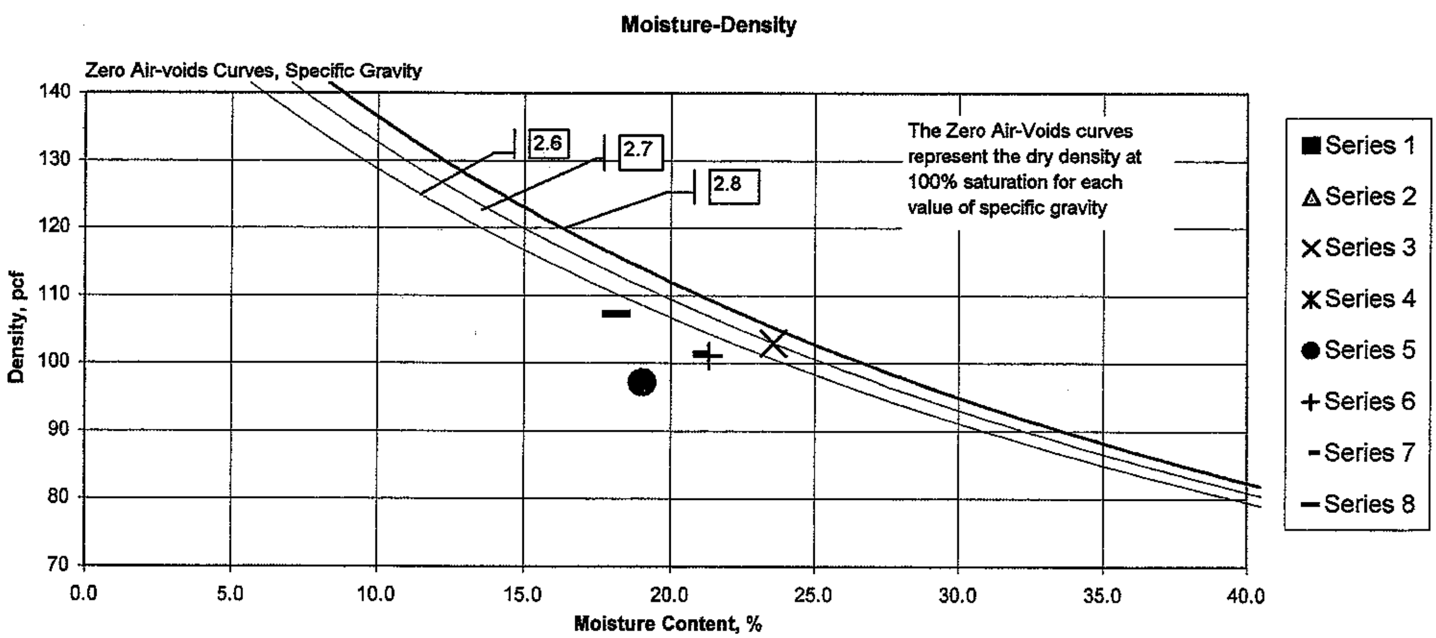


Moisture-Density-Porosity Report

Cooper Testing Labs, Inc.

ob No:	519-008b	Date:	07/22/04
lient:	4LEAF, Inc.	By:	MJ
roject:	Brentwood Rec. Ctr. - J148	Remarks:	

oring:	4B1	4B1	4B1	4B4	4B4	4B4	4B4	4B4
ample:	1-9A	1-9B	1-10	Bulk	4-1	4-3	4-4	4-6
epth, ft:	33.5-34	34.5-35	38.5-40	1-2	3-4.5	6.5-8	7.5-9	13.5-15
sual escription:	Brown Silty SAND	Brown Sandy SILT	Brown CLAY	Dark Brown CLAY with Sand	Brown Silty SAND	Brown Sandy Lean CLAY	Brown Sandy SILT	Brown Sandy CLAY
ctual G_s								
sumed G_s			2.70		2.70	2.70	2.70	2.70
tal Vol cc			226.48		149.62	183.25	218.15	221.83
ol Solids cc			138.19		86.19	109.85	130.90	140.98
ol Voids cc			88.29		63.44	73.41	87.25	80.86
oisture, %	25.3	32.3	23.6	19.2	19.0	21.3	20.8	18.1
t Unit wt. pcf			127.2		115.7	122.7	122.3	126.7
y Unit wt. pcf			102.9		97.2	101.1	101.2	107.2
turation, %			99.5		69.8	86.1	84.3	85.4
rosity, %			39.0		42.4	40.1	40.0	36.4
id Ratio			0.639		0.736	0.668	0.667	0.574
eries	1	2	3	4	5	6	7	8



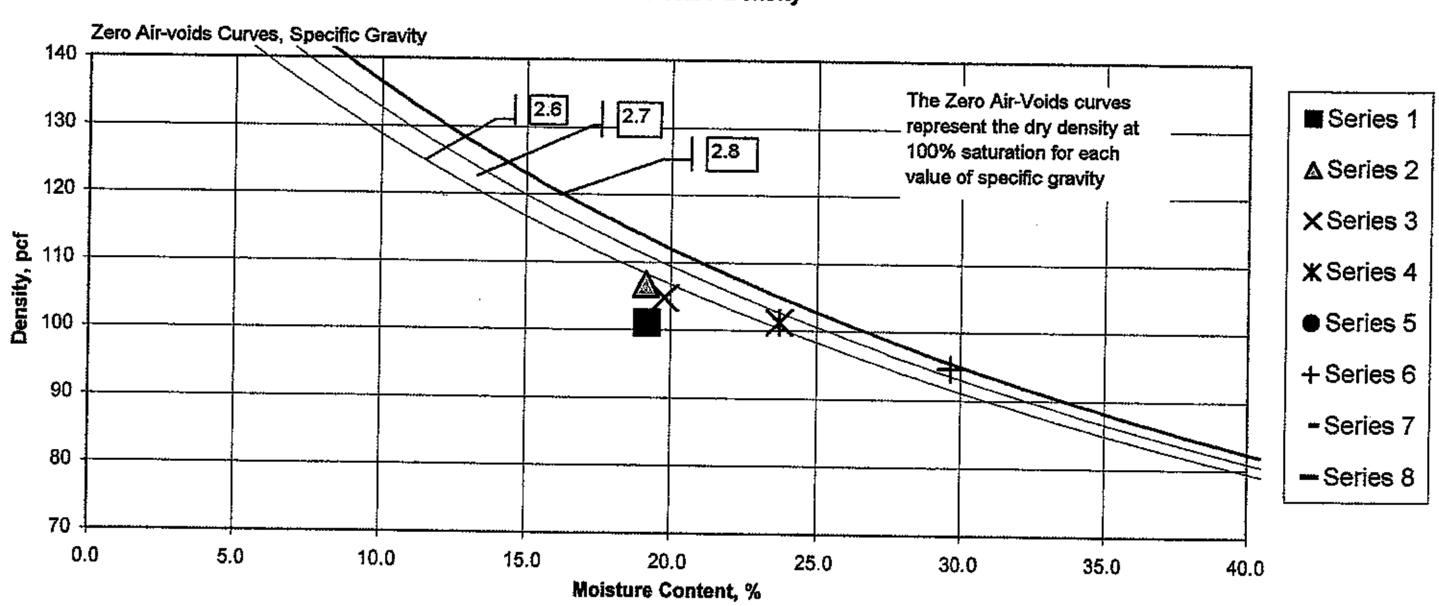
Moisture-Density-Porosity Report

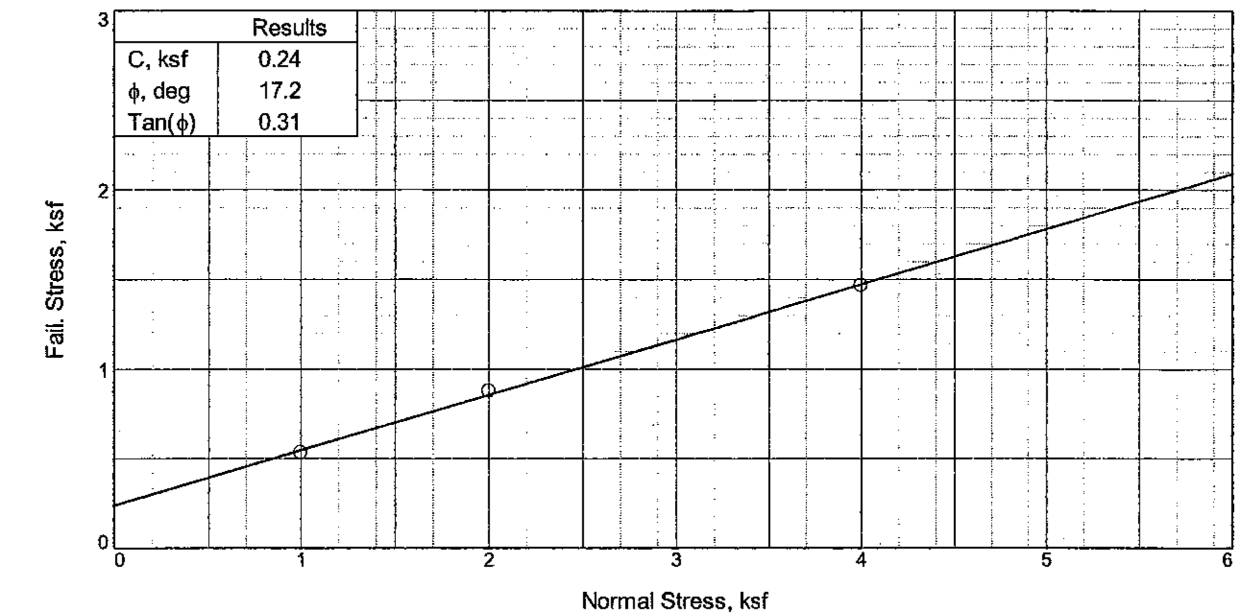
Cooper Testing Labs, Inc.

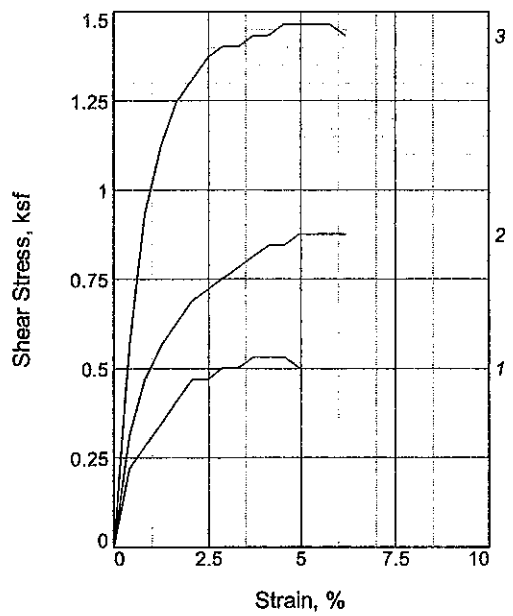
›b No:	519-008c	Date:	07/22/04
ient:	4LEAF, Inc.	By:	MJ
oject:	Brentwood Rec. Ctr. - J148	Remarks:	

oring:	4B4	4B4	4B4	4B4	4B4	4B4		
mple:	4-7	4-8	4-9	4-10	4-11	4-12		
pth, ft:	18.5-20	23.5-25	28.5-30	33.5-35	38.5-40	43.5-45		
sual scription:	Brown Sandy CLAY	Brown Sandy CLAY	Brown Sandy SILT	Brown Silty SAND	Brown Clayey SAND	Brown CLAY		
tual G_s								
sumed G_s	2.70	2.70	2.70	2.70		2.75		
tal Vol cc	225.55	221.83	225.55	225.55		228.17		
l Solids,cc	134.96	140.37	140.03	135.41		125.68		
l Voids,cc	90.59	81.47	85.52	90.14		102.49		
isture, %	19.2	19.1	19.8	23.7	25.7	29.6		
t Unit wt, pcf	120.3	127.2	125.5	125.3		122.7		
Unit wt, pcf	100.9	106.7	104.7	101.3		94.6		
turation, %	77.3	89.0	87.5	96.2		100.0		
rosity, %	40.2	36.7	37.9	40.0		44.9		
id Ratio	0.671	0.580	0.611	0.666		0.815		
ries	1	2	3	4	5	6	7	8

Moisture-Density







	Sample No.	1	2	3
Initial	Water Content, %	25.0	26.3	26.1
	Dry Density, pcf	94.0	93.4	95.6
	Saturation, %	85.2	88.4	90.0
	Void Ratio	0.7930	0.8042	0.7958
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	24.5	25.1	23.7
	Dry Density, pcf	97.9	98.8	103.9
	Saturation, %	91.5	96.0	99.9
	Void Ratio	0.7224	0.7057	0.6531
	Diameter, in.	2.42	2.42	2.42
	Height, in.	0.96	0.95	0.92
	Normal Stress, ksf	1.00	2.00	4.00
	Fail. Stress, ksf	0.53	0.88	1.47
	Strain, %	3.7	5.0	4.5
	Ult. Stress, ksf			
	Strain, %			
	Strain rate, %/min.	1.00	1.00	1.00

Sample Type: Undisturbed

Description: Brown SILT with Sand, slightly plastic

LL= **PL=** **PI=**

Assumed Specific Gravity= 2.7

Remarks: *DS-CU* A fully undrained condition may not be attained in this test.

Figure ________

Client: 4LEAF, Inc.

Project: Brentwood Rec. Ctr. - J148

Source of Sample: 4B4 **Depth:** 5-6.5'

Sample Number: 4-2

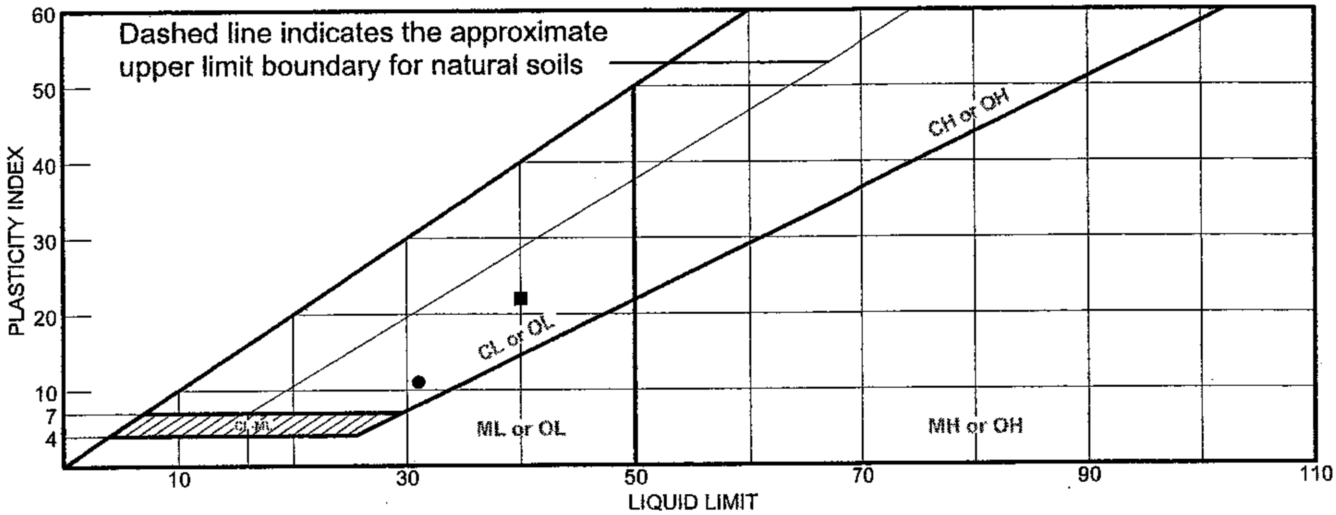
Proj. No.: 519-008 **Date:** 7/22/04

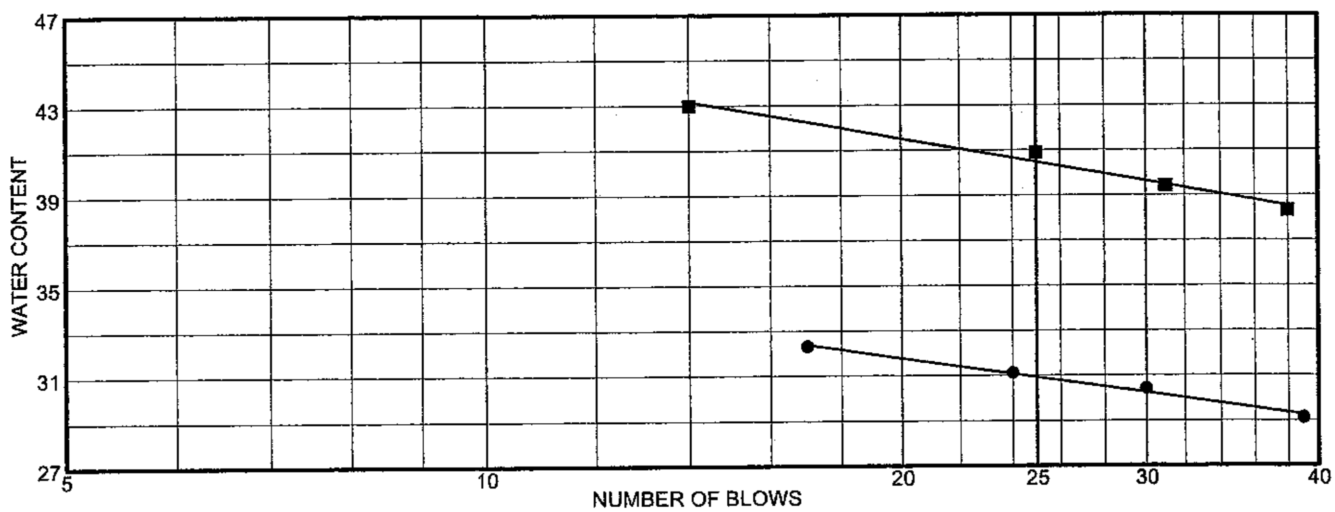
DIRECT SHEAR TEST REPORT

COOPER TESTING LABORATORY

Tested By: MD **Checked By:** PJ

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Brown Sandy Lean CLAY	31	20	11			
■	Brown Sandy Lean CLAY	40	18	22			

Project No. 519-008 **Client:** 4LEAF, Inc.

Project: Brentwood Rec. Ctr. - J148

● **Source:** 4B1 **Sample No.:** 1-1 **Elev./Depth:** 3.5-5'

■ **Source:** 4B4 **Sample No.:** 4-3 **Elev./Depth:** 6.5-8'

Remarks:

●

■

LIQUID AND PLASTIC LIMITS TEST REPORT

COOPER TESTING LABORATORY

Figure

COOPER TESTING LABORATORY

#200 Sieve Wash Analysis
ASTM D 1140

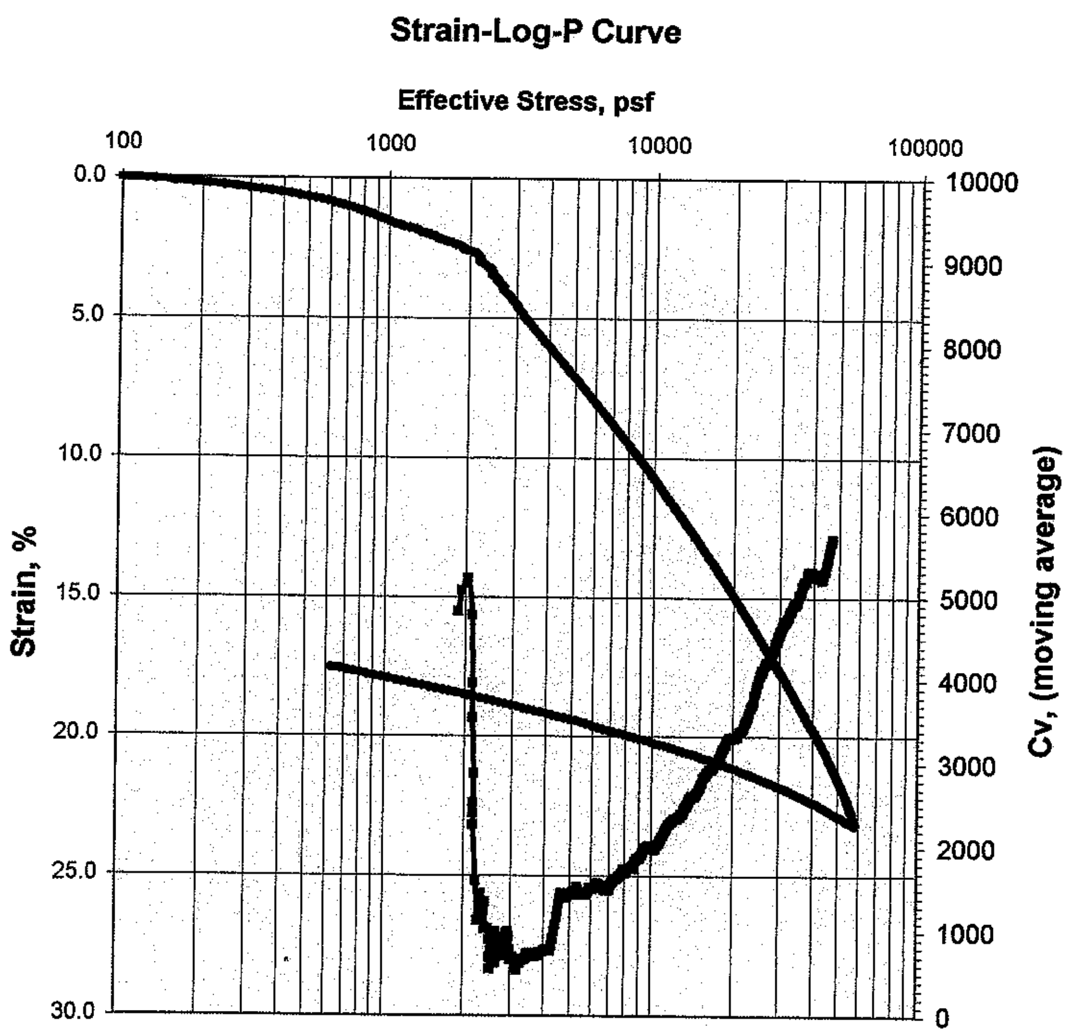
Job No.: 519-008 **Project No.:** J148 **Run By:** MD
Client: 4LEAF, Inc. **Date:** 7/22/2004 **Checked By:** DC
Project: Brentwood Rec. Ctr.

Boring:	4B1	4B4							
Sample:	1-2	Bulk							
Depth, ft.:	5-6.5	1-2							
Soil Type:	Brown SILT with Sand, slightly plastic	Dark Brown CLAY with Sand							
Wt of Dish & Dry Soil, gm	334.7	429.3							
Weight of Dish, gm	165.0	156.2							
Weight of Dry Soil, gm	169.7	273.1							
Wt. Ret. on #4 Sieve, gm	0.0	0.0							
Wt. Ret. on #200 Sieve, gm	28.3	47.3							
% Gravel	**0.0**	**0.0**							
% Sand	**16.7**	**17.3**							
% Silt & Clay	**83.3**	**82.7**							

Remarks: As an added benefit to our clients, the gravel fraction may be included in this report. Whether or not it is included is dependent upon both the technician's time available and if there is a significant enough amount of gravel. The gravel is always included in the percent retained on the #200 sieve but may not be weighed separately to determine

Constant Rate of Strain Consolidation Report
ASTM D 4186

Job No.:	519-008a	Boring:	4B4	By:	DC
Client:	4-Leaf	Sample:	4-5	Date:	7/23/2004
Project:	J148	Depth:	9-10.5'		
Soil Type:	Brown SILT				



Ass. Gs = 2.7	Initial	Final
Moisture %:	24.8	18.0
Density, pcf:	96.5	113.6
Void Ratio:	0.746	0.484
% Saturation:	89.7	100
Initial Back Press., psi	50.6	

Remarks:

Max Pore Pressure Ratio	3	PI =

Collapse Test Report
ASTM D5333

Job No.:	519-008	**Boring:**	4B1	**Date:**	7/22/2004
Client:	4LEAF, Inc.	**Sample:**	1-2	**Tested By:**	MD
Project:	Brentwood Rec. Ctr. **Proj. No.:** J148	**Depth, ft.:**	5-6.5	**Checked:**	DC
Soil Description:	Brown SILT with Sand, slightly plastic				

Load, psf	150	300	550	1100	2200	4400	4400		
Deformation, in.:	0.0023	0.004	0.0067	0.0119	0.0196	0.0337	0.1718		

	Initial	Final		Remarks:
Moisture Content %	10.9%	24.2%	**Load at Collapse, psf**	
Dry Density, pcf	83.3	100.5	**4400**	
Void Ratio	1.025	0.677	**% Collapse**	
Saturation %	28.8%	96.4%	**14.29%**	
Specific Gravity -	**Assumed:**	2.7	**Measured:**